

MINIMUM REQUIREMENTS FOR MUNICIPAL SOLID WASTE LANDFILL IN
THE STATE OF FLORIDA

By

AMMAR F. AL-TAQOUT

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Abstract of Thesis Presented to the Graduate School
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MINIMUM REQUIREMENTS FOR MUNICIPAL SOLID WASTE LANDFILL IN THE STATE OF KUWAIT

By

Aamer F. Al-Yaqoob

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Chairman: Frank Townsend

Major Department: Civil Engineering

The environment and human health in the State of Kuwait are threatened from improper Municipal Solid Waste (MSW)-disposed management. MSW landfills in Kuwait act as dumping grounds rather than safe disposal sites. Basic tools that assist in protecting the environment are not included in these systems. Such tools include a base layer, final cover, gas collection system, and leachate collection system.

In an effort to help Kuwait Municipality to build safe MSW sites that are environmentally acceptable and cost effective, laboratory and field tests were conducted on the natural subsurface layer of sand (gash) to study its applicability as base and cover systems. In addition, Polysar® software was used to study environmental migration through the gash layer. Furthermore, a high-density polyethylene (HDPE) liner that was exposed to the environment was tested for its tensile strength and test resistance in order to identify its workability in the field. Laboratory tests included permeability assessment using riser/drain tests and falling head tests, whereas field tests were conducted by two-stage

leachate permeability tests. Laboratory results indicated that gash sands are considered a low permeability soil when compacted at optimum moisture content and maximum dry density. Permeability values as low as 4.8×10^{-12} and 6.7×10^{-12} cm/sec were recorded by both lab tests. A field test program was conducted on both natural gash deposits and compacted gash pads. Permeability from the natural deposit suggested that gash deposits at great depths could be considered as secondary barrier system in the liner. Similarly, permeability values of compacted gash were between 3.8×10^{-12} and 1.17×10^{-11} cm/sec. Upon desaturation, permeabilities of compacted gash decreased by one magnitude. On the other hand, drying and wetting cycles conducted on the gash soil in the lab had increased permeabilities by one magnitude. In the field, gash can be compacted at 10% of maximum dry density but at moisture contents always dry of optimum. Gash compaction at dry of optimum may explain the minimal effect of cracking and shrinkage in the layer.

Analysis of Fullsaver® models show that gash will measure the magnitude of contaminant where concentrations diminish at 10-m depth of the gash layer after 200 years. HDPE testing pointed out that Young's modulus and strength of the liner decrease after long periods of field applications, and HDPE tends to accumulate moisture below its surface which keeps the gash liner at high moisture contents. The permeability values of the gash liner at the pad that consists of a composite liner systems had decreased after six months. Overall, compacted gash sands are recommended for use in Kuwait as liner and upper systems without HDPE liners. The liner system should include a leachate collection system, although Kuwait landfill classification model does not require these systems based on chemical water balance.

CHAPTER 1 INTRODUCTION

Background

The State of Kuwait lies at the northwest edge of the Arabian Peninsula at the head of the Arabian Gulf. Kuwait is an arid country with no surface water and very limited reserves of usable ground water. The rapid development of Kuwait in recent years has created environmental problems that can pose serious health hazards. Also, the Iraqi invasion in 1991 has caused disastrous damage to the environment, which eventually has threatened human health. In total, Kuwait's environment and the health of Kuwait citizens have been deeply threatened in just years. Until recent times, developing countries like Kuwait were not interested in solving environmental problems, confining their interests to social and economic development. But as these problems arise and pose serious environmental degradation and health hazards, the Kuwait government shifted some attention towards solving such issues.

A common problem in economically developing countries is the disposal of waste in a manner which frequently results in environmental degradation (Joshi et al. 1990). Waste management in Kuwait is one of the issues dragging the country into enormous environmental problems. Dumping waste in secondary landfills resembles the prevailing method of municipal solid waste disposal in Kuwait. An secondary landfill refers to a non-engineered facility for the disposal of municipal solid waste that is badly used,

leaking, damage, and operated with consequent adverse environmental impacts. It was estimated that 4.5 million tons of waste were received annually by active municipal landfill facilities in Kuwait with a total capacity of 483.2 million cubic meters already used and 136.5 million cubic meters currently planned in all municipal landfill facilities (QDA, 1996a). Almost 98 % of all solid waste generated in Kuwait during recent years was deposited in active landfills. Al-Maswari and Al-Jubouri (1994) reported that in 1996 the land consumption by dumping sites would increase at a rate of 0.227 square kilometers per year. Kuwait Planning Ministry (1996) estimated that the projected population at year 2008 would be 2 million people and at year 2013 would be 3.8 million people. Furthermore, in Kuwait the rate per person of solid waste generation is about 1.25 Kg per day (QDPT 1996, Koushki 1993). Each person in Kuwait generates more than 2.2 times as much waste as each person in Germany per day (Koushki 1993). The growth in population and the high rate of production of solid waste per person will result in the generation of an enormous amount of solid waste and increase the need to build residential areas next to old or new dumping sites.

The high technology of sanitary landfills did not reach Kuwait until now. Kuwait spends \$12 per ton collecting, transporting, and dumping municipal solid waste. In 1996 the government spent \$11.3 million to dump waste. The disposal of wastes in Kuwait frequently results in environmental degradation even with spending that amount of money. A logical approach to reduce such waste disposal problems is to establish a minimum requirement for landfill siting and design methods (Joshi et al. 1995). As a result, cost will be reduced and the environment and human health will be protected. For a small country like Kuwait, land value is precious for both the government and the

community. To save the land for future use and to conserve money, Kuwait should establish minimum guidelines that regulate waste management and increase the level of awareness for the future-designers of such waste-disposal practices in waste disposal.

Problem Statement

Municipal solid waste (MSW) disposal practice in the state of Kuwait creates a great threat to the country's environment and human health. The current practice of disposal is dumping waste into a group of sand quarries, on average depth of 5-18 meters, and then covering them with sand. Municipal solid waste disposal sites receive all sorts of waste materials like sewage waste, chemical waste, and debris. The different wastes are intermingled, with rarely a sorting procedure or a separation operation. Also, most of the wastes are exposed to the high temperature of Kuwait, and placed in steep slopes. The current disposal sites are operated by companies lacking any necessary engineering experience in terms of designing, operating, or maintaining environmentally safe landfill.

Waste disposed in Kuwait does not meet the minimum environmental criteria for Municipal Solid Waste Landfills (MSWLFs) that ensure the protection of human health and the environment. MSWLF waste in Kuwait are in dumping grounds rather than safe disposal sites. These units include none of the components that provide basic environmental protection, such as a leach line, final cover system, gas collection system, and leachate collection system (if needed). From the current position, contamination of the ground water table is possible due to the nearby location of the water table beneath the bottom of waste disposal sites. Because Kuwait has an arid climate, a relatively small amount of leachate will be developed in landfill sites. Nevertheless, an assumed minimum

of the amount of leachate will develop due to the improper disposal of liquid sewage and chemical wastes with municipal wastes. The lack of proper MSW guidelines that assure the protection of the environment and human health is a major problem for the Kuwait Municipality, the only government agency that deals with MSW disposal practices. The need for such guidelines that follow proper disposal practice is a must for the country's environment. These guidelines will enable Kuwait Municipality to implement safe disposal methods for new landfills and use the proper closure systems for the old ones.

The proposed research will not only control the waste practices, but also it will address the applicability of natural resources, such as Kuwait's impermeable calicheous layer of sand (Jilch), as leach and covers. Furthermore, the research will provide new specific guidelines that will apply to countries with similar climate. Finally, there is a great demand to protect the environment and human health. Therefore, a comprehensive study that defines the problem, develops practical solutions, and drafts a government document is very essential to Kuwait.

Research Purpose

The principal objective of this research is to help the Kuwait Municipality to build safe MSW disposal sites that protect the environment and human health. The outcome of the research will be based on field and lab tests that will be performed to achieve that goal. The research will shed light upon the applicability of using high-density polyethylene (HDPE) liners in Kuwait and calicheous sand (jilch) as a natural leach and cover. Furthermore, the research will study the effects of improper dumping practices on the environment, soil, and groundwater analytically by using contaminant transport

computer programs. This will lead to significant environmental and economic benefits. Environmentally, the soil will be protected from leachate contamination and the balanced migration of gases, and the groundwater will be protected from the long-term contamination that usually affects humans, plants and animals. Also, the air will be protected from undesirable gases by designing proper covers and gas venting systems. Economically, natural resources will be used as tools for constructing safe sites which will help in reducing the high cost of such containment systems. The overall product of the proposed research will place Kuwait's solid waste practices in the first stage of advanced technology for MSW containment sites.

Conclusion

Recognizing the problem, the objectives of the proposed research are the following:

1. Document and evaluate current conditions of present municipal waste dumping areas.
2. Study the effects of closed dump areas on the environment, soil, and ground water table using analytical techniques including pollution migration computer programs such as FATHOM5, a program that implements a solution to the one-dimensional dispersion-advection equation.
3. Investigate the applicability of using covered colcemant mud (gumbo) as a final cover system and check the viability, reliability of using possible types of natural and/or geosynthetic films that suitably Kuwait's environment.
4. Develop standards to draft guidelines for new waste disposal sites in terms of siting and designing liners and covers for Kuwait's environment.

Research Scope

The scope of the proposed project is to select the best tools and technology for the municipal solid waste industry that will suit Kuwait's soil environment. Field and lab tests were performed to achieve that goal. Moreover, recent software and computer models such as *Proland* that deal with the design and operation of landfills were tested using the conditions and parameters for Kuwait's landfill sites to give a better understanding of environmental migration. The listed investigations in this research emphasized collecting information related to the current waste situation such as:

1. The volume of solid waste generated in Kuwait.
2. Solid waste characterization: A detailed description of types of wastes dumped at disposal sites, and the percentage of each type in the total volume.
3. Current solid waste problems such as leachate migration, gas generation, improper dumping, and fire generation.
4. Government regulations and special requirements.
5. Availability of topographic maps.

The field and lab work will contribute to the engineering solution for the recommended guidelines for Kuwait's environment. Although the research will emphasize landfill design tools and techniques like liners and covers that are suitable for Kuwait, a complete concept of proper waste handling and management will be introduced in the prepared guidelines. The guidelines will include waste classification, design procedures, and recommendations for leachate management. The research result will be prepared for the State of Kuwait environment using all available data and information.

The coordination of climate and environment between Kuwait and other Gulf countries like Saudi Arabia, Qatar, Bahrain, Oman, and the United Arab Emirates will make the research results very useful and beneficial to MSW management for this region.

CHAPTER 2 LITERATURE REVIEW

History of Dumping Waste in Kuwait

Municipal Solid Waste (MSW) disposal in Kuwait has passed through different stages. Before the 1950s, MSW was dumped in the Arabian Gulf and on uncontrolled open land, particularly in the desert. From the 1950s to the early 1960s, disposal was in open-burning dumps (JCDK 1994). In the 1960s, the government had to construct a large amount of sand and gravel, creating undesirable quarries, for the construction of a highway network and sea front extensions due to the rapid population growth and cultural development. During that development era, Kuwait had started to replace municipal solid waste by the establishment of a fertilizer production plant. Unfortunately, the government had to close down the plant due to numerous operational and management problems (Al-Rumayh 1994). In the mid-seventies, Kuwait Municipality banned the disposal in open burning dumps and replaced it by a secure dumping method. In the late 1970s, waste was collected and transferred to assigned disposal sites, but rarely with supervision or control. In the 1980s, MSW was disposed in a group of quarries, average depth from 5 to 18 meters. These sites were the old group of abandoned quarries which had resulted from excavations prior to the oil era and the invasion. At that time, the government tried to establish fertilizer plant-operated and managed by the private sector

The plant was operated for a short period of time then due to financial problems the plant was shut down.

Kuwait Municipality had been the acting government agency that operated all MSW activities, disposal and collection from the past until the Iraqi invasion. After the Iraqi invasion, the government of Kuwait shifted to the private sector in a step to re-activate the economy and to allow the private sector to participate in the development and reconstruction of the country. The private sector was assigned to operate almost all government activities because of high expenses of operations that accompanied such activities, specially MSW disposal. The government was spending 140 million per year for the disposal of household refuse since (KBR 1994). At the present time, Kuwait Municipality assigns a company, selected through public tenders, for a period of two years to dispose waste at selected sites (Al-Bazzaz 1994). In the same fashion, different companies are assigned to collect and transport the waste to the dump sites.

Almost all companies that compete to win the contract for MSW disposal have little or no sanitary engineering practice experience in terms of designing, operating, or monitoring environmentally safe landfills. The main responsibility of these companies, as stated in a short document from the Committee of Public Control, Date number 1994, is to operate the dumping site by providing different landfill vehicles like Crawler Tractor, Rubber-Tired Front-end Loader, Water Systems, Earth Movers, and Drag Lines. Also, the companies should control the site by security checks and steel fences around the site boundaries. Furthermore, the contractor should take the responsibility of the following:

1. Excavating the road used to cover the waste;
2. Transferring the required sand to the dumping site;
3. Compacting the waste and covering it with sand;

4. Sprinkling water that is not used for drinking purposes on the waste twice a day.
5. Sprinkling manure on the waste once weekly.

These are some of the important specifications listed in the LHM document. It is clear that the guidelines listed above do not meet the minimum requirements to minimize the detrimental effects of a landfill on the surrounding environment during disposal or after closure. The lack of extensive regulations that protect human health and the environment, and ensure safe waste disposal practices in the mining loop in Korean MSW management during the past years.

Relevant, Completed, or Ongoing Research

At the present time, there are some attempts, headed by the Department of Environmental Affairs in Korean Municipality, to regulate the MSW practice in Korea by establishing specifications which suit the real circumstances of the country. The most serious attempt is a joint effort between Korean Municipality, the University of Florida represented by the author of this thesis, and KEO International-Consultants which is one of the leading engineering firms in Korea. Chapter 9 details this preliminary study and its potential impact for the future. The lack of such research is obvious since no published work was found or documented. Nevertheless, the Department of Environmental Protection in the Ministry of Health (DEPH), or HPA, reorganized four grants from the Resource Conservation and Recovery Act (RCRA), Subtitle D Program (40 CFR 129), and provided them as Environmental Guidelines for Household Refuse Landfills. These grants are the following:

1. Leachate Retention
2. Design Criteria
3. Site Management
4. Employees Health and Protection

The handling of the previous points was general with no implementation plan to protect Kuwait's environment. It was clear that this work will not change the practice since this department is not the acting regulator or agency that handles the MSW disposal. Furthermore, the absence of experience in Kuwait from those selected for MSW work played a major role in disregarding and adoption of the guidelines by Kuwait Municipality officials.

Research was devoted before and after the Iraq invasion to study waste resource generation and the potential for reuse and recycling (Al-Darwish et al. 1983, Al-Mutairi et al. 1987, DEPH 1994, and Kowalek 1995). Al-Mutairi et al. (1987) estimated that Kuwait generates about 2.5 million tons per year of solid and semi-solid wastes (Table 2-1). Approximately 80% of these wastes were organic in nature (Table 2-2) (DEPH 1994). MSW in the State of Kuwait was estimated to be about 1.7 million tons per year and approximately 80% of these waste were residential which is about 0.7 million tons per year (Table 2-3) (DEPH 1994). Al-Mutairi and Madhoun (1994) presented the average composition percentage of municipal solid waste based on comparison with three different previous studies (Table 2-4). The rate per person of solid waste generation is about 1.35 Kg per day (DEPH 1994, Kowalek 1995). Based on the previous studies, it was expected that the amount of residential solid waste generation in 1999 will be about 1100 tons per day which is about 0.4 million tons per year (DEPH 1994). The latest study on waste composition is presented in Table 2-5, which classifies the daily household solid waste, by type (Kowalek 1995).

Table 3-4. Solid Waste Generated in Kuwait (adapted from Al-Mashay et al. 1987).

Type	Quantity (Tons/year)
Food Waste	338,400
Wood	32,000
Cardboard & Paper	210,000
Glass	38,750
Textiles	30,000
Metals	65,700
Plastic	1,500
Rubber	1,700
Construction Materials	760,000
Chemicals	11,000
Lubricating Oils	41,000
Fluores	15,000
Industrial Greasy/oil Sludge	11,000
Domestic Sewage Sludge	30,000
Hospital Waste	5,000
Knaghtonness Waste	15,000
Drilling Mud	450,000
Combin	1,000
City Sludge	40,000
Total	2,485,500

Table 2-2 Organic Wastes Generated in Kuwait (adapted from ESM 1994)

Type	Quantity (Tons/year)
Food Wastes	150,000
Cardboard & Paper	150,000
Wood & Kindling	70,000
Agricultural Wastes	40,000
Animal & Poultry Manure	120,000
Municipal Sewer Sludge	10,000
City Sludge	50,000
Domestic Sewage Sludge	30,000
Lubricating Oil	40,000
Cellulose Exports	1,000
Milk Whey	2,000
Tires	1,000
Miscellaneous	50,000
Total	1,000,000

Table 2-3 Solid Waste Generated in Kuwait (adapted from DEPA 1994)

Type	Quantity (Tons/year)	Percentage (%)
Food Wastes	150,000	50
Paper	120,000	17.5
Cardboard	20,000	3.3
Plastic	60,000	10.0
Iron	10,000	1.6
Glass	10,000	1.7
Textiles	10,000	1.6
Wood	10,000	1.6
Miscellaneous	40,000	6.7
Total	300,000	100

Table 2-6. Average Percentage of MSW in Kuwait (adapted from Al-Murairi & Mubarek 1994)

Type	Percentage (%)
Food and Vegetable	38
Paperboard, and carton	27
Plastics	8
Textiles	3
Metals	8
Glass and ceramic	4
Wood	11
Paper	N/A
Miscellaneous	3
Total	100

N/A, not applicable data

Table 2-8 Types of Daily Household Solid Waste (adapted from Kaudia 1995)

Type	Quantity (Tons/day)	Percentage (%)
Food	1255	31.9
Paper	434	11.1
Plastic	314	8.1
Glass and Metals	121	3
Glass	111	2.8
Miscellaneous	179	4.4
Total	3915	100

Landfill water treatment and gas migration studies were very rare in Kuwait. The only two research studies that attempted to focus on these issues warned of expected approaching dangers. Al-Masroum and Mohamed (1994) investigated the extent of pollution in soil of the primary landfill after the Iraq occupation, when large quantities of solid wastes were generated during that period. Field sampling and analysis programs were performed, and the concentrations of nutrients and metals were calculated. Kuwait Institute for Scientific Research (KISR) conducted the second study in cooperation with the Environmental Protection Council. The study was performed on an old dumping site, named Al-Qurain, a group of sand quarries from the 1930s later used as a dumping site to dispose different kinds of wastes from 1936 until 1983 when it was closed. Soil borings, waste characterization, leachate samples, and gas emanations were the core elements of Al-Qurain research. Political pressure was the main factor leading to this investigation because the Al-Qurain area was planned to be the largest industrial project in the 1990's. Overall, the studies showed that Kuwait is going in the right direction in refusing waste disposed by reuse and/or recycling (DEPH 1994, KISR 1994). Nevertheless, Kuwait certainly is going in the wrong direction for waste disposal and waste management. Also, the policy for reuse and/or recycling is moving in very slow steps toward solving the real issue of reducing waste and minimizing the amount of waste disposed.

International Practice For Designing Healthy Landfill

Sanitary landfills have been the most economical and environmentally acceptable method for the disposal of municipal solid waste throughout the world. Technologies of

of (1993) defined sanitary landfills as an engineered facility for the disposal of MSW designed and operated to minimize public health and environmental impacts.

Two of the main criteria used in international practice for constructing sanitary landfills are design criteria, and closure systems. In design criteria, a mandatory physical separation between the waste body and the ground water region is applied to all new MSW landfill units and lateral expansions. The separation consists of a composite liner and a leachate collection system to remove leachate from the landfill and minimize the potential of ground contamination. In the U.S., the leachate collection system must be constructed to maintain less than a 30-cm depth of leachate over the liner. Also, the composite liner must consist of an upper component of a minimum 30-mil Flexible Membrane Liner (FML) or a 60-mil High-Density Polyethylene (HDPE), and a lower component of a minimum 60.8-cm thick compacted soil layer with hydraulic conductivity no greater than 1×10^{-7} cm/sec. In Germany, the composite liner must consist of an upper component of synthetic liner, preferably HDPE, with a minimum thickness of 2.5 mm (25.4 mils), and a lower component of a minimum 33-cm thick compacted clay layer with a hydraulic conductivity no greater than 1×10^{-7} cm/sec (Saltschek 1994). Table 2-4 presents the different design criteria for the liner systems from different international regulations. Also, Figure 2-1 illustrates the components for the bottom lining systems for municipal solid waste landfills.

Closure and post-closure care modules for all MSWLF units do have a final cover system. The main purpose of the cover system are the following (Toblerold et al. 1994, Massmann et al. 1998, and Tscheringhaus et al. 1999):

1. Minimize infiltration of water into the waste and residues

1. Limit the uncontrolled release of landfill gas
2. Uplift the ground surface and provide suitable slopes for runoff and drainage of surface water
3. Isolate the buried waste from contact by humans, animals, and vegetation, which eventually reduce the potential for fires

The final cover system must be composed of an erosion layer underlain by an infiltration layer. In the U.S., the erosion layer consists of a minimum of 15-24-in. of surface material capable of retarding the growth of native plants. The infiltration layer must be composed of a minimum of 45-33-in. of surface material that has a permeability of less than or equal to 1×10^{-6} cm/sec, or less than or equal to the bottom liner system permeability or natural subsoil permeability, whichever is less. The USEPA (1992) recommended an alternative final cover design for non-hazardous waste landfills. The recommended cover includes a gas vent layer, drainage layer, 24-in. PMD, or 60-in. HDPE, leachate layer, and filter layer along with infiltration layer and erosion layer. The filter layer, generally consisting of cobbles, is used to prevent the penetration of deep-rooted vegetation and burrowing animals (Glenn & Swagston 1996). Table 3-2 presents the different design criteria for the cover systems from different international regulations. Also, Figure 3-2 illustrates the components for final cover systems for municipal solid waste landfills.

Another design parameter for modern sanitary landfills in the U.S. and throughout the world is controlling gas, which is produced from the microbiological decomposition of organic waste. Landfill gas control systems are established to prevent unwanted movement of landfill gas into the atmosphere or surrounding areas. The movement of landfill gases is controlled in numerous atmospheric reactions that occur continuously, to

prevent gas migration, and to allow recovery of energy from methane if possible (Tabatabaieghani et al. 1990).

Therefore, the theory of current landfills is simple: keep the waste and all by-products away from any contact with the environment. However, the technology used for siting, designing, building, and operating is fairly complex.

Environmental Agencies Roles in Kuwait

Municipal Waste Management is the responsibility of Kuwait Municipality. The two departments that deal with MSW at Kuwait Municipality are the Cleaners Department, and the Environmental Affairs Department. The responsibilities of the two departments are the following:

Cleaners Department:

- 1- Supervise and implement the management conditions in the document from the Committee of Public Control (date number 1/16 for the municipal waste disposal).
- 2- Supervise and pursue the collection and transportation of waste to the dumpsites by the assigned private companies.

Environmental Affairs Department:

- 1- Perform the different environmental studies that concern the human health and environmental protection.
- 2- Prepare a master plan for Kuwait Municipality programs that implement environmental protection.
- 3- Help Kuwait Municipality Departments establish their programs with regard to saving the environment.

Table 2-6: Lining Systems for Municipal Solid Waste Landfill (adapted from Manacore et al. 1996, EPA/AF 1994, Bolintine et al. 1994)

Legend R = required NR = not required P = permeability D = drainage	Liner Components					
	Compacted Soil		GM Layer	Geo-Textile	Geo-Membrane	Leachate System
	Thickness	K ^a				
	Country	z (cm)	k (cm/sec)	z (cm)	Placement	
USA	60-72	1x10 ⁻¹¹	43	N	R	R
UK	100	1x10 ⁻¹²	-	R	N	R
Portugal	100	1x10 ⁻¹²	-	N	R	R
Germany	75	1x10 ⁻¹²	50	N	R	R
Hungary	80	1x10 ⁻¹²	50	N	R	R
Italy	100	1x10 ⁻¹²	-	N	R	R
France	50	1x10 ⁻¹²	50	R	R	R
Belgium	100	1x10 ⁻¹²	-	N	N	R
Japan	60	1x10 ⁻¹²	50	R	R	R
South Africa	60	1x10 ⁻¹²	13	N	N	R

Table 2-7: Cover Systems for Municipal Solid Waste Landfill (adapted from Manacore et al. 1996, EPA/AF 1994, and Bolintine et al. 1994)

Legend R = required NR = not required O = optional P = permeability D = drainage	Cover Components				
	Compacted Soil		GM 2-12	Geo-Membrane	Grass Layer
	Thickness	K ^a	Thickness	Placement	Thickness
	z (cm)	k (cm/sec)	z (cm)		z (cm)
Country	z (cm)	k (cm/sec)	z (cm)		z (cm)
USA	43-72	1×10^{-11}	O	O	15-24
Germany	50	1×10^{-12}	50	R	100
Italy	-	-	O	N	100
South Africa	43	-	13	N	30

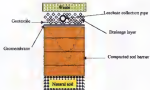


Figure 2-1 Leachate System for Municipal Solid Waste Landfills

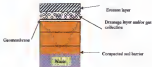


Figure 2-2 Cover System for Municipal Solid Waste Landfills

Another environmental agency that just recently dealt with MSW issues is the Environmental Public Authority (EPA). Kuwait EPA tasks are to deal with the protection of Kuwait's environment by leading research, proposing solutions, and drafting general rules. The role for Kuwait EPA as defined in the existing legislation is to assist the ministry tasked protecting the environment by serving as the lead-office for environmental issues. Also, The Ministry of Health is considered one of the oldest agencies that had been involved with different environment matters through the Department of Environmental Protection (DEPH). The previous department, in general, published most of the early studies on waste composition and waste management. Recently, DEPH duties were transferred to Kuwait EPA as a move to minimize interference in tasks between the Ministry of Health and Kuwait EPA, and to benefit from the experience of DEPH staff.

For the Oil Industries Area, Shuaiba Area Authority, there has been a department that manages the area in terms of environmental protection. This agency is called the Environmental Protection Center, and its duties are as follows:

1. Investigate the transboundary causes and reasons of pollution in the area.
2. Set up short and long-term measures to mitigate pollution problems arising from the existing industries.
3. Review environmental impacts of industrial expansion to be recommended in the area.

On the other hand, the Council of Ministers has formed under its subject the Highest Council for Environmental Protection headed by the first deputy of the Prime Minister. The tasks of the Highest Council are to adopt environmental standards laws

and agreements and to implement these agreements in Kuwait. Also, the Highest Council serves as the supreme environmental authority in the country to protect human health and the environment.

In summary, there are many agencies that are competing between each other to protect the environment and human health. This abundance of governmental branches may cause an institutional interference between them. Although some of these departments have the energy and ambition to solve the current environmental issues, they are faced by different challenges due to their new roles in the field which derived from other, older governmental agencies that dealt with the environment.

Government Attitude and Public Awareness

Waste Management is a concern to the government in Kuwait. This concern is evident from the numerous meetings with the officials in the Department of Environmental Affairs at Kuwait Municipality, Environmental Public Authority, Department of Environmental Protection in the Ministry of Health, Kuwait University. All officials stated that the government should act quickly and efficiently to resolve this issue. Although Kuwait was spending about 40 million U.S. dollars per year on the disposal of household refuse alone and now is spending about 14 million U.S. dollars, many government officials hesitate to spend that amount of money on safe and secure MSW landfill. The Kuwait Municipality Council, which oversees all the funding to solid waste projects, lacks the ability to fund and approve a safe ecological solid waste landfill due to the absence of a decision-based model which illustrates the following:

1. The problems, present and future impacts on human life and the environment, and the importance for future safe disposal solutions
2. A well-defined regulation that suits Kuwait's environment and complies with the technology of MSW problems in the developed countries.
3. A complete description for economic saving with environmentally sound integrated waste systems, operational and effective plans for future investment and profit from the industry.

Politics and economics are the major issues that continuously play a role in shifting from sub-standard practice to a safe technology. The Department of Environmental Affairs in Kuwait Municipality is acting positively to direct the Kuwait Municipality Council to consider the options available for municipal waste disposal practice.

Public attitudes toward the problem of waste disposal practices are still not very powerful compared to other developed countries. Sharma and Sangaria (1994) stated that the increasing public awareness of environmental issues has resulted in various federal and state environmental regulations in the United States of America. Lack of general knowledge about the danger that faces the public and the environment from recent waste disposal practices is the main factor for the Kuwaiti people. Public concern usually comes from individual groups in certain locations that have been affected by the disposal process. For example, people reacted positively to stop the disposal in the Al-Jahra Al-Shaykhia site because of foul odors and a nest of insects and animals that accompanied the sanitary method from the increase in volume of waste arriving at the site during the Iraq occupation and the post-liberation period. Also, in 1993 public outcry increased in the Al-Qurna housing project. The project was constructed next to a waste area of old landfills that caused disgusting odors, gas emissions, and spontaneous fires. Al-Qurna residents

forced their representatives to act and raised concerns both in the Congress and in the Municipal Council. The latest complaint was in April 1995, when the congressmen explicitly mentioned the methane gases that caused odor at the southern part of the Qurna Area.

Public awareness toward environmental issues should be increased by education. Public power to affect the political decisions is normally weak. Also, since Kuwait is considered a small country, public policy should always be effective and fast even if the waste site is not in your backyard. The people should understand that their children would be affected from the dumpsite in the future. The rapid growth in building land use, and the government is still forced by law to place an citizen in Kuwait. The people in Kuwait can influence their congressmen, from both the Kuwait Congress and Kuwait Municipal Council, to pass laws that require immediate solutions for environmental problems.

Case Studies

The purposes of introducing these case studies are to explain the disastrous outcomes from not following the proper methods for municipal solid waste disposal and to illustrate the importance of monitoring leachate and gas generation at the landfill before and after closure. Two case studies are introduced in this section: Al-Qurna disposal site in Kuwait, and the Land Reclamation Company Landfill in northeastern Wisconsin in the United States of America.

Al-Qurain Disposal Site

From 1906 to 1983, the Al-Qurain Site was used to dispose municipal solid waste in an area appraised to be 175000 square meters of old sand and gravel quarries (Figure 3-3). Many different wastes were disposed in the site such as food waste, industrial waste, and debris (Figure 3-4). Al-Ramahi (1994) reported that waste disposal at the Al-Qurain site was performed randomly without any protection to the waste by sand layers upon which led to pile up the waste in large quantities. No records exist on the quantity of waste disposed at the site nor if intermediate fill was used during the filling process (Tetra Tech 1998).

The problem began when the Public Housing Agency (PHA) in 1983, received the approval from Kuwait Municipality (KM), including maps and records for the location of the disposal site, to build government provided houses in the Al-Qurain Area. PHA performed preliminary soil borings on the site that showed waste in locations other than specified by KM and in much greater depths. Although the problem was obvious from the start, PHA continued the plan of constructing houses due to political pressure. Later PHA discovered, during construction, that the disposal site locations had been supplemented from Kuwait Municipality were inaccurate. Through the process of avoiding the locations of waste disposal, many disposal sites were partially removed by the contractors. Furthermore, at the boundaries of the housing project, massive holes, gas emission from biological decay, and spontaneous fires were noticed (EPC 1992).

In the early 1990s, a team from the Department of Environmental Protection in the Health Ministry and the U.S. Environmental Protection Agency conducted field tests on the Al-Qurain area by drilling 20 pits at the dumpsite.

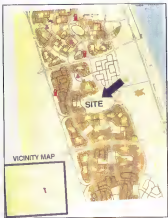


Figure 2-3 Al-Qurayn Abandoned Landfill Site (adapted from Tera Tech 2009)



Figure 2-4 Waste Layers in Al-Qurayn/Dump Site after PMA Construction

In 21 pits, the explosive susceptibility for gases created was 100%. Also, methane levels were more than 2% in 12 pits and 30% in two pits. The presence of methane in the air in concentrations between 5 and 13 percent is considered explosive (Takahashi et al., 1993). Additional soil borings were performed on the soil under the waste or in between to assess the effect of waste on the soil. The borings estimated the amount of waste in the site to be about 3 million cubic meters, and the depth of the queries where the waste were dumped was from 3 to 18 meters. Figure 2-3 illustrates the surface area and depth of each waste query.

The team included appropriate solutions to treat the problem. These solutions were described as following:

1. Total removal of waste to different location
2. Static engineering solution

The first solution was eliminated because of bad odors, emission of gases, and possible fire generation that would affect the transportation area in the site or in the surrounding environment. Also, the cost for this solution was about 28 million U.S. dollars, which was considered a high cost compared to the other solutions. The second solution was most acceptable which was controlling the gas emissions with safe environmental methods. The cost for the second solution is estimated at approximately 19.5 million U.S. dollars. The Ai-Qumia area is considered one of the largest public housing projects for the government. It was estimated that 12,000 units were built in the area. The vital issue in this study is the fact that no one at the middle 1990 and no solutions have ever been implemented. At the same time 1,500 houses with around the waste area were located within 30 meters from the dumpsites and already occupied with families.

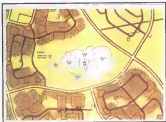


Figure 3-5 The surface Areas and Mean Depth for Waste Quarries in Al-Qurayn Arm (adapted from Terna Terna 1999)

This problem is a real example of bad waste management in which the lives of the people are threatened and the money of the country will be wasted for remediation.

Land Reclamation Company (LRC) Landfill

The Land Reclamation Company (LRC) Landfill is located in southwestern Wisconsin. The landfill was built in accordance with the Wisconsin Municipal Solid Waste Regulations, which is much stricter than the RCRA Subtitle D Program (40 CFR 238). In early March 1990, LRC detected methane at five gas monitoring locations along the East Side of the landfill. Because of the close proximity of private residences, and according Wisconsin Administrative Code, LRC proceeded to investigate the extent and reason for gas migration. Steps to reduce the problem that were taken in March and April 1990 were the following:

1. Compile hydrogeologic and geologic data from appropriate existing boring logs, groundwater level monitoring data, and different well details (i.e., gas/achute, extraction wells, leachate land wells, gas monitoring wells, etc.)
2. Perform mass results of monitoring for gas concentrations at 24 wells in the area of methane detection to clearly define the area of migration.
3. Conduct a series of borehole probes to further define the lateral extent of gas migration.
4. Monitor for gas in the sparsely building region in the landfill gas migration area.
5. Perform two geotechnical borings and several test pits in the path of migration.

The result of the investigation shows that methane was being detected at perimeter gas probes. Several reasons were believed to be the cause, such as, untreated

hydrogeological and climate conditions, absence of a gas impermeable barrier between the landfill and the adjacent property line, and ineffective gas extraction from the existing internal system along the east slope of the landfill. In early July 1993, the following design plan was implemented:

1. Installed new gas wells and abandoned ineffective existing wells.
2. Installed gas and leachate conveyance pipe to connect new wells and the horizontal negative control trench.
3. Constructed a compacted clay barrier wall. Figure 2-6 and 2-7 illustrate the slope before and after design.
4. Installed the gas probes inside and outside of refuse limits.
5. Connected the new extraction wells and horizontal negative control trench to the existing gas main header.

By September of 1993, the remedial design was completed successfully. The results from the remedial actions were the following:

- Volume of gas extracted has increased.
- Gas probes inside the landfill confirm the effectiveness of the replacement gas wells.
- Gas probes on the property boundary show no further migration of methane.
- Landfill gas odor has been reduced or eliminated.

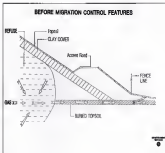


Figure 3-6 The Land Reclamation Company Landfill before Migration Control Features (adapted from Freese and Larson 1997)

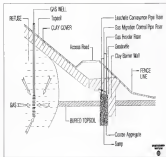


Figure 3-7 The Land Reclamation Company Landfill after Migration Control Features (adapted from Free and Lanyon 1997)

The total cost for the project was 540,000 U.S. dollars in which \$450,000 was for investigation, \$700,000 was for construction, and \$110,000 was for Quality and Assurance.

CHAPTER 3 LANDFILL PRACTICE IN KUWAIT

Current Practice for Waste Disposal

The current practices in disposal sites in Kuwait are unsafe for humans and for the environment. The sites receive all kinds of wastes such as food wastes, oil products, debris, agricultural wastes, chemical materials, and sewage wastes. Although the sites have some employees from the contracted company or from Kuwait Municipality, there is no obvious control on the sites. Dumping can be performed as easily as throwing a trash from your car at most locations. Illegal night-dumping is a usual practice for waste owners if officials did not accept what you wanted to dump during daytime. Lack of security and responsibility are obvious at these sites. Karama (1996) noted that the landfilling processes should be selected in a fashion which maximize the accommodation capacity of a landfill, select the method of construction in accordance with the volume of waste to be carried into the landfill site, and to minimize the harmful effects on the surrounding environment either on human or on the ecosystem. Neither of the previous objectives of landfill management has been implemented in Kuwait's waste disposal sites. The sites have many operation and management problems, which can be described briefly by the following points:

- 1- All the sites were not designed for dumping wastes
- 2- No permit is granted to the private owners at any time- As a result the operator has no obligation in protecting the environment

- 3 Absence of clear and distinctive task plans
- 4 Improper disposal strategies
- 5 Lack of order and security
- 6 Absence of constructed roads to the disposal area
- 7 Lack of sufficient equipment and tools
- 8 Presence of incompetent and insufficient employees at the site

The process of waste disposal at these sites is usually to dump the waste as close to the quarry as possible, and then a compactor vehicle, if available, with the rear door or ramp. Usually compaction is not performed. After that, the Taliban-Talud Front Field Leader pushes the waste to the quarry until the ground level is reached. At the end of day, the waste is covered with a thin layer of sand. Many parts of the waste pile, especially the sides, are not covered periodically due to the steep slopes. This procedure is causing more bad odors, and self waste combustion due to high temperature and humidity in the country. Most of the wastes are exposed for a long time without any protection. When the desired elevation of the quarry is reached, sand is spread above the waste directly. A cover that includes an erosion and sedimentation layer is not defined in Kuwait's waste management vocabulary. Also, slope stability of neither the waste nor the sand cover is considered. Failure of the quarry or the waste poses a great threat to the site attendees and employees. One warning system or barrier-collection system have been never installed at the sites, except at the Tulaibiyah site where a few long capped PVC pipes were seen above the ground surface. From these remarks, the observer can imagine the aspect of the problem Kuwait is facing environmentally. Figures 3-1 to 3-4 illustrate the conditions in a number of disposal sites in Kuwait.



Figure 3-1 Garage Waste Truck Dumping Polluted Waste in the Dumping Site

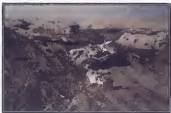


Figure 3-2. Food Waste, Debris, and Agricultural Waste Dumped in the Sewage Waste Quarry



Figure 3-3 Oil Waste Product Mixed with Different Kinds of Waste



Figure 1-4 Exposed Paper, Cardboard, and Different Types of Waste on the Dumping Site Without Any Cover

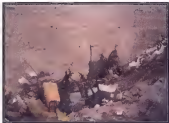


Figure 3-5 Agricultural Wastes Exposed to the Environment on a Steep Slope

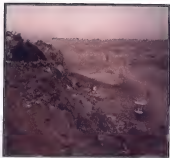


Figure 3.46 A View of the Quarry and the Uncontrolled Water Works as a Flooded
Passageway at the Bottom of the Quarry



Figure 3-2 Slaughterhouse Waste Dumped Without Any Protection



Figure 3-4 Cars at the Bottom of the Quarry Where Waste Films Are Disposed at Steep Slope

Waste Disposal Sites

There are thirteen listed waste disposal sites in Kuwait. Al-Qayla, Al-Jahra, Angara, Al-Qurtan, East Sabahiyah, Jleith Al-Sagayrah, Mirsalih, Nahr Abidiah, Seventh Ring Road Site (North and South), Shuaiba, South of Sabhan, Sabahiyah and Wafra. Figure 3-8 displays the old landfill sites in Kuwait. Dates of opening and closing operations at the sites, sizes, and types of waste that had been received by these landfills are listed in Table 3-1.

Domestic Waste as defined by the Cleaning Department (CCDP) includes residential, industrial and commercial wastes, trees, wood, tires, plastics, furniture, cardboard, paper materials, steel, electrical appliances, sponge products, floor sand from construction, animal residues, slaughter house wastes, and bulky wastes. Moreover, business medicine and chemicals, entertainment products, and even gamma hospital wastes are disposed as domestic waste with the permission of the Environmental Protection Agency (CCDP 1998). On the other hand, debris waste includes both construction and demolition waste, and liquid waste. Construction and demolition wastes consist of concrete blocks, sand, bricks, painting materials, steel plumbing and bathroom equipment. Liquid waste includes wastewater from areas not connected to the main sewage system, sludge produced from treatment plants, slaughterhouse wastes, dairy factory products, brick factory waste (white sludge), food factory waste, and refinements refinery wastes.

Many of the listed landfills such as Mirsalih, South of Sabhan, Jleith Al-Sagayrah, and Shuaiba were closed upon the request of different government agencies. Before the Iraq invasion, Jleith Al-Sagayrah was the main site in Kuwait. After

liberation, the site received all the war waste disregarding the waste type because at that time closing Kamut was the main objective. CDFP (1996) estimated more than 2 million tons of waste had been received at Al-Shaykhah site since the liberation until the site closed at the end of 1992. The Farwahan County forced the site to cease operation after increasing complaints from residents. Middle and South of Babylon were closed for security purposes by the request of the Ministry of Defense. So, within a short period of time the waste volumes had been transferred to different parts of the country rather than making it to the old site.

Recently, Kamut Municipality has allowed some of the current sand quarry owners to dump construction and demolition wastes on their sites to fill the quarries to the required levels instead of using sand. This permit permitted the illegal use of waste disposal that really benefited the owners of the sand quarries. Currently, no supervisors from KMA are located in these sites. The integrity of the owners and employees is the only assurance that only construction wastes will be dumped at these sites. A fee of six dollars is taken from each truck for disposing waste at these private sites. Although the government landfills have no disposal fee, many private companies like to dump elsewhere for many reasons, like shorter travel distances and freedom to dump without restrictions. A site visit, completed in April 1998, showed that all kinds of debris and wastes were dumped at these private sites, and a considerable number of leaking oil drums were dumped on the site without any concern for the oil leaking (Figure 3-10). Furthermore, piles of elemental sulfur were seen placed at the site. The tour indeed confirmed that the site is disastrous.



Figure 1-4 Old Landfills and Their Locations in Kuwait



Figure 2-19 CM Landing Drone Scattered Above Sledge Wreck on a Private Land²²

Table 3-1. Current and Old Waste Disposal Sites (adapted from CDMP 1999, and AJ Kamath 1994)

Landfill Site	Area (sq. km)	Date of Operation	Date Closed	Type of Waste
Al-Qayyah	NA	NA	NA**	Debris
Al-Jahra	28	NA	NA**	Domestic Waste
Al-Jahra	0.3	1988	*	Debris
Al-qura	NA	NA	NA**	Domestic Waste
Al-Qurayn	0.07	1978	1983	Domestic Waste
Tamr Salibiyyah	NA	NA	NA**	Debris
Malayh Al-Qayyah	1.28	1985	1993	Domestic waste
Musallab	0.3	1983	1995	Domestic waste
Mina Abdulla	1	1981	*	Domestic waste
North Seventh R.R.	8	1988	*	Debris
Musallab	NA	1984	1991	solid waste
South of Salman	NA	NA	NA**	Debris
South Seventh R.R.	1	1982	*	Domestic waste
Salibiyyah	13	1987	*	Domestic waste
Wadi	NA	NA	NA**	Domestic Waste

Legend

NA: not applicable

* Active Operation

** Closed Site

Current Sites Conditions

At present, there are five landfill sites in Kuwait that are operated and managed by Kuwait Municipality. These existing sites are listed as follows:

- North 7th Ring Road
- South 7th Ring Road
- Mina-Abdulla
- Salibiyyah, east
- Jahra

Figure 3-11 shows the locations of these existing landfill areas. Al-Ramadh (1994) stated that the total number of domestic waste trips per day to the main municipal waste disposal site was 300. According to CIOF (1998) the number of domestic waste trips per day was increased 60 percent in 1995. Furthermore, the average total number of domestic waste trips to the landfills was 368 trip per day. The purposes of studying these sites are to investigate the level of performance of the dumping sites, evaluate landfilling procedures, and to decide based on observations, the basic needs for operating and designing future landfills. Table 3-2 illustrates the evaluation criteria for the current operated landfill sites by Karach Municipality.

North 1st Ring Road Site

The North 1st Ring Road Site is currently operating as a construction waste disposal site. During the site inspections in July 1997 and March 1998, the following observations were noticed:

- 1- There is no weighing station, fence or borders, or entrance gate.
- 2- Unpaved roads leading to the trenches.
- 3- Unwashed concrete is disposed all over the site without any controlling measure causing bad odors.
- 4- Large quantities of the waste are scattered over the ground surface instead of being dumped in the quarry.
- 5- All types of construction and demolition wastes are disposed in the quarry without any sorting or inspection.



Figure 2-11. Planning Landfill Sites Locations in Kuwait

Table 5-3. Site Evaluation for Existing Landfill Sites in Kansas

Evaluation Criteria	Landfill Sites				
	Site No. 1	Site No. 2	Site No. 3	Site No. 4	Site No. 5
Entrance gate	C	F	F	F	F
Site fencing	C	F	F	F	F
Weighing station	F	F	F	F	F
Water inspection	D	D	D	D	D
Site office	C	C	D	F	F
Paved roads	F	F	F	F	F
Explosive signs	D	D	D	D	D
Site plan	F	F	F	F	F
Qualified employees	D	D	D	D	D
Safety equipment	F	F	F	F	F
Communication system	F	F	F	F	F
Site storage containers	F	F	F	F	F
Methods of disposal	F	F	F	F	F
Scavenger prevention	C	F	F	F	F
Waste classification	F	F	F	F	F
Scraping, overexposed	F	F	F	F	F
Waste compaction	F	F	F	F	F
Waste prevention by water	F	F	F	F	F
Waste open burning ban	D	D	D	D	D
Sludge stability	F	F	F	F	F
Early cover	D	D	D	F	F
Leach system	F	F	F	NA	NA
Leachate collection system	F	F	F	NA	NA
Gas collection system	F	F	F	NA	NA
Gas monitoring system	F	F	F	NA	NA
Leachate monitoring system	F	F	F	NA	NA
Final cover	F	F	F	F	F
Surface drainage control	F	F	F	F	F
Traffic control	F	F	F	F	F
Waste and land control	F	F	F	F	F
Fire protection	F	F	F	F	F
Truck washing facility	F	F	F	F	F
Waste disposal method	F	F	F	F	F
Waste-control measures	F	F	F	F	F
Special waste handling	F	F	F	F	F
Water quality monitoring	F	F	F	F	F

Legend: A = Excellent, B = Good, C = Fair, D = Bad, F = Not exist, NA = Not applicable

South 7th Ring Road Site

Work at this site started in 1992 following suspension of disposal operations at the Jaber Al-Khayssah site. The site is currently operating as a municipal solid waste disposal site. Nevertheless, all sorts of wastes are being received such as oil products, sewage waste, chemical materials, debris, etc. Al-Karadah (1994) reported that the Seventh Ring Road has received not less than 1,000 tons per day. The CDEP (1994) reported that the monthly average quantity of municipal solid wastes collected at the South 7th Ring Road site in 1993 was approximately 28,648 tons (343,780 tons/year). This represents a 15% increase from the amount reported in 1994 for this site (248,850 tons/year). The site serves the capital area and its suburbs. The following observations were recorded during the site visits in July 1997 and March 1998:

1. There is no weighing station, fences or barriers, or entrance gate.
2. Waste is exposed for long periods of time.
3. Oil products were mixed with the waste.
4. No compaction was performed on the waste.
5. Waste was piled in a steep slope.
6. Waste materials were randomly collected inside the pits.

Mutan-Abdulla Site

The wastes received at this site are both debris and domestic. The site serves mainly the southern region of Kuwait, including most of region of Wafiq. According to CDEP (1994), the yearly quantity of wastes received at this site in 1994 and 1995 were 113,348 and 149,888 tons respectively. The following conditions were observed during inspection visits in July and December 1997:

1. Wastes are dumped in a random fashion into spaces averaging 3-m in depth.
2. Liquid wastes were dumped in trenches next to the other types of waste. The waste was a mixture of sewage, oil, and multichlorinated fatory products.
3. There is no weighing station.
4. Disposed wastes are not covered properly nor compacted.
5. The overall site has bad odors most likely caused by liquid wastes.

Subayiyah Site

The Subayiyah site is currently operating as a municipal waste disposal site. The site is located next to the heavily populated Jabra residential area. So this site receives wastes collected from the northern parts of the country. According to CDEP (1998), the yearly quantity of wastes received at this site in 1994 and 1995 were 423,958 and 326,115 tons respectively. This is the only site that has a defined entrance and fences at the front-side. Also, there are recording activities but no weighing station. The reason for the decrease in the amount of waste between 1994 and 1995 is due to the start of operations of a private fertilizer plant. The following conditions were observed while inspecting the site in March 1998:

1. A great number of seagulls are scavenging and flying over uncovered wastes.
2. Much of the wastes are disposed over the ground surface instead of being deposited in trenches.
3. The overall site has bad odor, most likely caused by the sewage wastes.
4. Liquid wastes are commingled with all types of waste.
5. Wastes are dumped into the quarry without any compaction.

Jabra Site

The Jabra site is used for construction and demolition waste disposal. The following notes were recorded during site visit in February and March 1998:

1. Most of the wastes are disposed irregularly on the ground surface.
2. The traffic in the site was very congested and unorganized.
3. There is no weighing station, no fences or entrance gates.
4. The roads leading to the quarry are unpaved and unstable.

Waste Composition

Municipal solid waste (MSW) generates leachate and then passes through the processes of decomposition and percolation through the waste. The wastes disposed in landfills may consist of many different materials such as non-degradable inorganic materials, rapidly biodegradable organic materials, and slowly biodegradable organic materials. In order to design waste processing facilities, to develop landfill classification systems, and to estimate the amount of gas that can be generated, accurate data on the quantity and composition of waste materials are required. Components that typically make up MSW and their relative distribution among different countries are reported in Table 3-3.

In total, Kuwait generated about 5.2 million tons of solid and liquid wastes in 1994 and 1995. In 1996 the amount of waste generated was reduced due the opening of the Organic Fertilizer Company, which receives about 700 tons per day of waste. Unfortunately, the factory had a fire in the middle of June 1996 that forced it to cease operations. Table 3-4 presents the quantities of each waste source from 1993 until 1996. Clearly, the waste composition shows that most of the components could be managed in a more useful method other than dumping. According to CDEF (1998), the only sludge that is collected in special containers at the landfill and then stored in steel oil. Figures 3-12 to 3-14 show MSW components as weight percentages in 1994, 1995, and 1996 respectively. On average, 40% of the waste is organic in nature. Based on previous studies and the data available, MSW in Kuwait consists 8% food wastes, 3% paper and cardboard materials, and 3% plastic materials. The total amount of food wastes generated in Kuwait may range between 480,000 tons per year to 310,000 tons per year. Liquid

water and sludge quantities pose about 46.7% of the total waste disposed, which should be considered when designing leachate collection systems or liner systems unless different disposal methods are adopted. Construction and demolition waste, which includes construction debris, construction site debris, excavated material, and road construction debris makes an average 40.1% of the total waste disposed in landfills.

Table 3-3: MSW Components in Weight Percentages for Different Cities in the World (adapted from Minomura 1986, and Bakewell 1994)

Components	City					
	U.S.A. New-York	Greece Athens	China Fujian	Turkey Istanbul	Germany East side	Korea Pohang
Paper/No materials	30	39	45	61	34.01	74
Paper & cardboard	21	19	5	16	11.32	12
Plastics	N/A	7	1	3	6.61	5
Glass	4	2	1	1	12.41	4
Textiles	N/A	N/A	N/A	3	3.73	N/A
Leather, wood, & rubber	3	4	1	8	N/A	N/A
Metals	3	4	1	2	5.49	3
Others	44	5	46	14	20	2

Legend:

N/A, Not Applicable

The Department of Environmental Affairs (1996) reported that hospitals, medical centres, and private laboratories generate 14 tons per day. Most medical wastes are dumped with household wastes.

The average daily generation of solid waste in Kuwait were 2,636, 3,775, 4,614 tons per day in 1994, 1995, and 1996 respectively. Based on this data, the average daily quantity of solid waste per person is 1.46 kg per person per day. Kowalski (1995) reported that the average size per person for household wastes is 4.3 kg per day.

Table 3-4: Waste Quantities (in 1,000 tons) Delivered to Municipal Waste Facilities in Kuwait (adapted from CDEF 1996)

Waste Type	1993	1994	1995	1996
Household waste, bulky waste and Household like Commercial waste	823.3	854.7	798.20	867.25
Excavated material, construction debris, road construction debris	15.8	19.5	2017.3	1940.7
Solid and semi-solid municipal wastewater sludge	NA	1256	923.8	1009.4
Liquid waste	NA	1864.5	1760.7	1563.4
Non-recycling rubbish food items	NA	8.2	9.1	9.32
Refrigerator oils	NA	0.392	0.412	0.3
Flammable liquids	NA	0.345	0.333	0.383
Total	-	5,092.4	3383	4691.35

Legend:

NA: Not Applicable

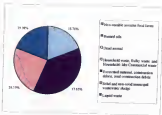


Figure 3-12 MSW Components as Weight Percentage for Korea in 1994

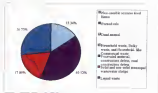


Figure 3-11 MSW Components in Weight Percentages for Kuwait in 1993



Figure 3-14 INCE Components as Weight Percentage for Kuwait in 1998

Gas Generation and Landfill Composition

The amount of landfill produced, over the long-term, is a function of the water balance at the surface of the landfill. Nevertheless, there are other factors that play an important role in the production of landfill especially in dry climate countries which practice improper waste management. These factors are listed as the following (Ham 1997):

1. Wetting characteristics of the waste
2. Water present in the waste when landfilled (including any liquid waste)
3. Horizontal flow through the sides of the fill
4. Channeling
5. Vertical upward flow through the bottom
6. Water taken up or produced by decomposition, and
7. Water loss through evaporation or leaching in landfill gas

Al-Salayteyah site

Landfill samples from Al-Salayteyah site were collected and analyzed for a period of three months (Al-Mutairi and Mohamed 1998). The water table at this site is located at a 6-meter depth below the waste level. The site has been in operation since 1982. Table 3-1 shows the Al-Salayteyah landfill characteristics from a downstream borehole. The pH data reported ranged from 6 to 9. The changes in pH are probably due to dilution with groundwater. This pH range suggests that waste is in the methanogenic phase, final decomposition phase, where methanogens convert acetic acid and hydrogen to methane and CO_2 (Ham 1997). The average BOD (Biological Oxygen Demand) value for the three months period was 114 mg/l and the corresponding average COD (Chemical Oxygen Demand) was 174 mg/l. Elmag (1993) reported that the BOD and the COD

range between 26-950 mg/l and 586-4,308 mg/l respectively for leachate in the metropolitan plant.

Nitrogen and phosphorus are major concerns in MSW leachate because of their importance in biological treatment (Kane and McClosley 1982). The total nitrogen includes ammonia, nitrite, oxidized ammonia, nitrate, and nitrite, which are often indicators of sewage, fertilizers, nitrogenous ammonia, phytates, and drugs (McIlhenny et al. 1999). Also, nitrite poisoning in lactating animals, including humans, can cause serious problems and even death (Pruay et al. 1982). The average concentration of ammonia in the leachate samples was 302 mg/l and the nitrite concentration was 24 mg/l. Elang (1989) reported that the range of ammonia and nitrite in leachate were 50-8,000 mg/l and 5-1-50 mg/l respectively. Phosphorus concentrations were higher than the ones recorded by other researchers like Elang (1989) and Kane and McClosley (1982) (8-1-83 mg/l). The average concentration of phosphorus was 60-14 mg/l. In comparison to other parameters, phosphorus concentrations from other upstream and downstream facilities were quite low. The tendency for phosphorus to adsorb to soil particles limiting its movement is suspected for the low concentrations (Kane and McClosley 1982, Pruay et al. 1982).

Hardness in MSW leachate is a measure of the presence of calcium, magnesium, iron, and possibly zinc (Kane and McClosley 1982). The average concentration of calcium hardness recorded at the Al-Sulaybiyah site was 5,388 mg/l, which is general is consistent with the typical range of 500-15,000 mg/l reported by Kane and McClosley (1982). Average alkalinity as MgCO_3 was 4,864-4-mg/l. Total alkalinity is a measure of the high buffering capacity, attributed in part to pH indicators.

Table 3-8 Leachate Samples From the Al-Kalaybaya Site Dewatered (adapted from Al-Mazloum and Muehler 1994)

Parameter	Date					***
	2-24-92	21-3-92	3-7-92	18-3-92	4-4-92	
pH, unit	8	8.7	8.6	8.6	8.9	6.5-8.3 ^a
BOD ₅ , mg/l	380	540.0	340.0	340.0	460.0	—
COD _{Mn} , mg/l	420	980.0	930.0	870.0	890.0	—
Total carbon, mg/l	590	971.0	971.0	540.0	641.0	—
Total inorganic carbon, mg/l	340	264.0	240.0	360.0	271.0	—
Total organic carbon, mg/l	250	707.0	730.0	180.0	370.0	—
Total dissolved solids (TDS), mg/l	28	27.0	14.1	30.0	31.0	300 ^b
Turbidity, unit	381	383.0	380.0	360.0	349.0	—
Conductivity, m/m	58	470	31.0	31.0	34.0	—
Total hardness, mg/l	1660	1230.0	1660.0	1170.0	1340.0	—
Calcium hardness, mg/l	708	3600.0	480.0	730.0	430.0	—
Magnesium hardness, mg/l	148	870.0	620.0	540.0	700.0	—
Alkalinity as MgCO ₃ , mg/l	303	683.0	543.0	243.0	304.0	—
Dissolved Oxygen, mg/l	1	1.0	1.0	1.0	1.4	3 ^c
Chloride, mg/l	208	280.0	280.0	210.0	195.0	250 ^d
Ammonia, mg/l	193	280.0	210.0	240.0	199.0	4.5
Nitrate, mg/l	28	12.0	180.0	23.0	13.0	10 ^e
Phosphate, mg/l	29	87.0	23.0	30.0	83.0	—
Sulfate, mg/l	1	1.0	1.0	3.0	1.1	250 ^f
Total sulfides, CFU/100 ml	390	280.0	280.0	340.0	280.0	—
Ca, mg/l	2880	13070	16070	18470	17200	200 ^g
Mg, mg/l	179	26350	15870	34400	36600	180 ^g
V, ppb	2259	27360	14310	28420	27310	—
Cr, ppb	19	310	3.0	23.0	42.0	0.01 ^h
Ni, ppb	380	240.0	240.0	83.0	220.0	0.1 ^h
Pb, ppb	29	490	17.0	21.0	60.0	—
Cd, ppb	181	803.0	121.0	489.0	410.0	5 ^g
Fe, ppb	522	875.0	490.0	749.0	812.0	6.2 ^g

Legend:

*** = Primary and Secondary Drinking Water Standards

^a = 40 CFR Parts 141 and 143

^b = The World Health Organization European (1967)

^c = The World Health Organization International Drinking Water (1964)

^d = New Mexico Regulations

low percent obtained in the readily-leach index of MSW landfills, average concentration was 6,496.5 µg/l. This high concentration of iron is not consistent with that reported for methanogenic phase. Heng (1987) reported that iron concentration in the methanogenic phase ranged from 2 to 250 µg/l. Excessive concentrations may be attributed to contaminated soil dumped, untreated sludge, oil waste products, and other industrial wastes. Moreover, the average concentrations for vanadium (V) and nickel (Ni) were 2,411.8 and 2,85.2 µg/l respectively. The presence of such quantities of V and Ni in landfill samples was due to soil pollution from motor waste oil (Al-Masman and Al-Masman 1994).

Al-Qurayn

Seven soil samples were collected and analyzed from soil layers located below waste layers at the Al-Qurayn site. The soil samples were found to be highly contaminated when compared to other soil samples far from pollution sources. The site was in operation from 1976 until 1983 and the waste cells at that site is located 30-meters below the waste level. Table 3-6 presents Al-Qurayn soil characteristics from samples below the waste layers.

The average pH recorded was 7.7, which is slightly lower than the value recorded at unpolluted sites. It is suspected that the highly buffered nature of the MSW landfills would tend to cause variation of the pH within a narrow range (Kmetz and McKinley 1982). The average concentration of total alkalinity was 2,300 ppm compared to 480 ppm at unpolluted sites, and ammonia concentration was as high as 89.4 ppm compared to 3.1 ppm. Toxic quantities of heavy metals such as iron, lead, zinc, chromium, nickel were comparable to the unpolluted sites except iron and zinc which were recorded in

consistently high concentrations. The flux average concentration was 15,000 ppm and zinc was 679.4 ppm. These high concentrations of zinc and iron indicate that the area had received different types of industrial wastes.

Methane (CH_4) and carbon dioxide (CO_2) gas concentration levels were measured at different depths inside 24 waste boreholes at the Al-Qusair site. It was found that in 13 boreholes, the level of gases at explosion susceptibility was 100%. Methane concentrations levels were more than 2% in 13 boreholes, and more than 50% in two boreholes (EPC 1985). When methane is present in the air in concentrations between 5 and 15 percent, it is explosive (Tabatabaieghani 1991). In general, the results also showed that gas explosion susceptibility increases as the sample selected comes from lower levels in the waste layers (EPC 1985). CO_2 concentrations levels were more than 40% in five boreholes and more than 15% in 14 boreholes. These levels of CO_2 indicate that the microorganism plays a controlling role in decomposition where CO_2 concentrations levels are approaching 45 to 50%. Concentration levels from selected boreholes for CH_4 and CO_2 at Al-Qusair site, thickness of waste layers, explosive levels, date of samples, and the sample depth below the ground surface are provided in Table 3-7.

Table 3-6 Al-Qusayr Soil Parameters from Samples below the water layer (adapted from EPC 1985)

Parameter	Mean & standard deviation of concentration (ppm)	
	Al-Qusayr site	Unpolluted sites
pH, unit	7.7 ± 0.18	8.58
Total Alkalinity	3280 ± 13380	48000
Sulfate	56178 ± 6384.4	32819 ± 8513
Phosphate	146.3 ± 80.9	15000
Ammonia	89.4 ± 31.0	3.18
Organic Nitrogen	354.9 ± 244.8	-
Cadm. Pb	19008 ± 27598	1821.4 ± 183.3
Lead, Pb	304.3 ± 636.3	188.3 ± 198.9
Cadm. Zn	679.6 ± 1809.4	21.6 ± 4.8
Chromium Cr	34.4 ± 23.8	24.6 ± 5.3
Nickel Ni	178 ± 34.3	362 ± 6.8

Table 3-7 Concentration Levels for Gases Measured at Al-Qasr Landfill Northside (adapted from EPC 2010)

Monitor	Leak Threshold	Date	Sample Depth	Explosion Level	CH ₄	CO ₂
#	(%)	Day	(Meters)	(%)	(%)	(%)
Q1	10	16-Jan	1.5	100	1.8	12.7
		16-Jan	2.5	100	2.0	100
Q5	40	11-Jan	2.5	100	2.1	13.0
		21-Jan	4.5	100	30.7	> 15.0
Q5	11.3	16-Jan	2.5	100	1.6	2.0
		21-Jan	11.5	100	10.9	> 15.0
		23-Feb	11.5	100	44.6	40.3
Q4	14.3	16-Jan	9.5	100	14.3	> 15.0
		21-Jan	9.5	100	7.6	12.3
		23-Feb	9.5	100	33.9	30.1
Q5	15.0	16-Jan	9.5	100	18.2	> 15.0
		19-Jan	16.5	100	13.9	> 15.0
		21-Jan	11.5	100	16.9	> 15.0
		23-Feb	16.5	100	30.1	40.3
Q4	5.0	11-Jan	9.5	100	16.2	> 15
		23-Feb	1.5	100	11.2	40.9
Q1a	4.0	11-Jan	2.5	100	21.8	> 15.0
		11-Jan	7.5	100	30.9	> 15.0
		19-Jan	7.5	100	16.7	> 15.0
		21-Jan	7.5	100	10.5	> 15.0
		23-Feb	7.5	100	47.6	40.6
Q1a	8.0	11-Jan	10.5	100	18.2	> 15
		20-Jan	10.5	100	6.6	7.6
Q1a	7.0	15-Jan	4.5	100	1.6	19.1
		21-Jan	9.5	100	8.0	0.1
Q1a	5.3	21-Jan	2.5	100	9.9	30.1
		23-Feb	7.5	100	7.9	36.6
Q1a	8.0	16-Jan	3.0	100	23.9	36.6
		20-Jan	10.5	100	1.1	1.8

Legend:

NR = Not Recorded

Kuwait Municipality Waste Management

Kuwait Municipality (KM) is the only agency responsible for waste management in the State of Kuwait. In recent years, KM assigns the disposal of MSW to one of the private sector companies hired to manage KM disposal sites. KM contracts the company for a period of two years with a possibility of a six-month extension. Bid N 176 is the most recent contract offered by the Committee of Public Contract Bids. Item 2 from the general conditions in the contract introduces the objectives of the bid, which is the disposal of MSW in the assigned sites according to the conditions and requirements stated in the contract. N 176 requirements clearly do not present the sufficient tools for managing disposal sites in a manner that protects the environment and human health. General requirements of site operation, engineering methods, personnel and financial measures will be discussed briefly in this section.

Operation Guidelines

The disposal company has to adhere to the special requirements supplied with the main N 176 bid. These special guidelines are the only source in the contract which inform the contractor of KM needs for waste disposal site management. The guidelines consist of sixteen items. Fifteen of these items deal with operation requirements and one item can be considered as design criteria. The 15 items are described below:

1. Site needs of equipment and vehicles and the contractor responsibility of providing mobile offices at sites are listed in Items 1-3 and 12.
2. Items 4 and 15 require providing special mobile lighting generators for night disposal. Items 3 and 7 allow KM to use the equipment and vehicles in other than the specified sites, and prevent the contractor from using, recycling, or selling any of the wastes except with written permission.

1. *Operating plan that specifies, in addition, detailed instructions for the daily operation of the landfill, contingency operations for waste handling in case of emergency, method and response of filling waste, and operation of gas, leachate, and overwater controls.*
2. *Records of weighing incoming waste*
3. *Detailed handling of hazardous wastes if identified by random waste inspection*
4. *Placement of waste methods that specify the required thickness bottom and after compaction and waste slope stability requirements*
5. *Daily and intermediate cover plans*
6. *Excision control measures, dust control methods, and fire protection and fire-fighting capabilities*
7. *Inspection plans including frequency, personnel, and training programs for the landfill employees*

Engineering Criteria

The operator is obliged to design and operate gas venting systems for the old waste sites and the current waste sites as stated in item 16. The system consists of 2 to 8 inch diameter plastic pipes with half-inch thickness. Lengths of these pipes range between 8 to 18 meters. Direction to set the gas venting system was described, without figures, as follows:

1. *Place perforated plastic pipes vertically on the last layer of waste and pouring a light cement mix to hold it*

2. The paper shall be covered to prevent rain from going inside the paper.
3. One meter shall be the height of the paper above ground surface.
4. The distance between the paper shall be up more than 100 meters.

It is to be noted that the landfill engineering perspective is lacking many of the different systems of MSW landfill design that cause a public nuisance or create a potential hazard to public health, welfare or the environment. These systems are briefly summarized as the following:

1. Liner system that serves as a protective layer beneath the solid waste. The liner shall be composite or double with a minimum hydraulic conductivity of no more than 1×10^{-11} cm/sec.
2. Leachate collection system which shall incorporate a piping collection network composed of perforated pipes.
3. Closure and post-closure plan that includes the installation of a final cover. The final cover shall be composed of an infiltration layer with a minimum hydraulic conductivity of no more than 1×10^{-11} cm/sec, and an erosion layer.
4. Detailed waste-monitoring program that includes sampling and analysis methods.

3. Stormwater management system constructed and maintained to prevent overflow from peak discharge for 25 or 50 year storm event.
4. Landfill gas monitoring and control systems designed to prevent explosions, fires, and to minimize off site odors.

Financial and Financial Assurance

According to RCWA 90A, site operators must submit 20% of the total contract amount as assurance for illegal acts and improper management. Most of the penalties and fines stated in the contract are for late performance of various tasks such as vehicle and employee absences, missing operations, stopping mandatory sprinkling, not abiding to the general requirements. The fines range from \$15 to \$1,000 per day.

Although 20% of the total amount is a guarantee for the site's entire life period, most international regulations require financial assurances for the stages after operation terminates at the disposal site. The operator must have a detailed written estimate of the cost of hiring a third party to perform the following:

1. Closure
2. Post-closure care
3. Closed phases of the site, and
4. Corrective Action

In Summary, the current and old waste disposal practices in Kuwait are leading the country to serious environmental degradation, and are threatening the safety and health of its citizens. From the waste composition section, it is concluded that waste products are not considered a useful raw material for various industries. Furthermore,

current disposal guidelines do not achieve the minimum requirement for protecting the country's environment and human health.

CHAPTER 4 ENVIRONMENT EFFECTS ON NFW PRACTICE

Climate

The state of Kuwait falls climatically within the desert zone. The climate of Kuwait is described as very hot, dry summers with an average daily temperature of 40°C (103°F) and mild to cool winters in which temperatures fall to as low as 4°C (39°F). The temperature is highest in July and August and lowest in January and February. Figures 4-1 and 4-2 illustrate the monthly average maximum and minimum temperatures in Kuwait respectively.

In Kuwait, rainfall occurs mainly from November to April. The annual average total amount of rainfall for the years 1954-1967 was 113-mm (4.4 inches). Rainfall ranges from a high of 242.4-mm in 1956 to a low of 21.3-mm in 1964. The mean recorded for the last five years was 144.6 mm (5.6 inches). In January, when the largest amount of rainfall usually occurs, a mean of 23.4-mm (0.9-inch) was recorded for the period between 1954 to 1966 (Figure 4-1). On the other hand, evaporation dominates most of the year. In August 1956, the highest average evaporation recorded was 183.1-mm. In January, evaporation reaches its lowest levels (3.5-mm). Figure 4-4 shows the variation of evaporation recorded in 1956 and 1967.

Dust storms, lasting at times for several days, occur mostly during the months of May, June, and July. A total of 46-days of dust storms was recorded in 1962. Strong dust

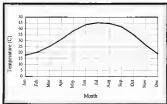


Figure 4-1 Monthly Mean of Minimum Temperature in Rawat (1962-1997)
(adapted from Meteorological Department, 1997)

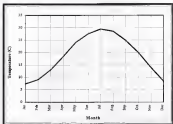


Figure 4-2. Monthly Mean of Minimum Temperature in Kuwait (1963-1997)
(adapted from Meteorological Department 1997)

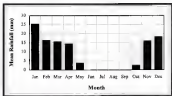


Figure-4.3 Monthly Mean of Total Amount of Rainfall in Karsna (1958-1997)
(collected from Meteorological Department, 1997)

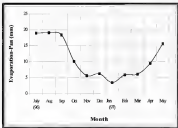


Figure 4-4. Evaporation-Pan Data Recorded in Karsin (1986-1997)
(adapted from Meteorological Department (1997))

storms usually cause visibility problems and shortens of breathing. For the past three years, only five dust storms have passed through the country. The northwest frequent winds, with a maximum speed of 24 m/s, are cool in winter and spring and hot in the summer. Southeastern winds spring up between November and March (Meteorological Department 1997).

The maximum relative humidity occurs in December and January, with a mean of 82 and 87% respectively. Summer relative humidity is the lowest, ranging between 34% to 47% for maximum, and between 7% to 17% for minimum. Figure 4-5 represents the monthly mean of maximum relative humidity between 1962 and 1997.

Hydrology

The main characteristics of Kuwait's hydrological state is the discrepancy of potential and actual evaporation compared with available precipitation. As a result, the total rainfall is neither sufficient for groundwater recharge, nor to support sustained flow of streams or rivers (Fouad et al. 1984). There is hardly any runoff of rainfall to the sea, and precipitation mostly evaporates or percolates into the ground. The sources of drinkable and industrial water are seawater that is processed in desalination plants, groundwater and to a limited degree, wastewater. Kuwait has been a pioneer in desalination program since 1949. The majority of Kuwait's fresh water comes from desalination plants (Yousif 1984). Although the majority of groundwater systems in Kuwait are brackish, the second most important source of usable water is groundwater. The capacity of the principal water fields producing brackish water in Kuwait is about 675 million-liters per day, and for fresh water only 4.4 million-liters per day (Yousif 1984).

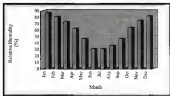


Figure 4-8 Relative Humidity Monthly Mean in Kuwait (1963-1997)
(Adapted from Meteorological Department, 1997)

Water Table Location

Water table depths in the vicinity of Kuwait City vary from 1.5 to 10 meters (Yousif 1984). The major source of this high rise in water table includes excess water from watering gardens, parks, yards, and leakage from water supply and sewage lines (Yousif 1984, Kennedy et al. 1993). Also, the presence of the impermeable sand layer that prevents water from percolating downward causes the groundwater to stop near the surface and rise in some locations. Outside Kuwait City, the water table depth may range between 12 to 15 meters (Ministry of Defense 1977). Nevertheless, in some locations outside Kuwait City the water table was found at 4 to 14 meters below the ground surface especially at the sand-spurges in the south of Kuwait.

Aquifer Systems

The demand for water draws special attention to groundwater resources, since Kuwait has no real rivers. Kuwait has two main aquifers, Kuwait Group and Dammam Formation. The systems within the Kuwait Group and that of the Dammam Formation form a mutually connected groundwater system (Jassabi 1967). Groundwater in aquifers of the Kuwait Group and the Dammam Formation is generally brackish to saline. The water salinity is around 3,000 mg/l in the south western and the north-western parts of the country, which gradually increases towards the eastern and north-eastern parts of the country with concentrations reaching a level of 150,000 mg/l near the coastal areas (Mubikpally et al. 1994). Fresh drinkable water is limited to the northern location of the upper part of Kuwait Group (Dibdiba formation), which is sometimes considered as an independent aquifer. The hydrogeological and stratigraphic subdivisions of the aquifer system of Kuwait are shown in Figure 4-6. The natural

discharge from the system is by vertical upward seepage into the Arabian Gulf and Shatt Al-Arab basins, where the direction of flow is from southeast to northeast, and by evapotranspiration in the areas where ground levels are close to the surface (Maidment *et al.* 1994, Younis 1994). A hydrological section of the direction of flow in the aquifer systems is presented in Figure 4-7.

Kuvait Group is a multi-layered aquifer system with three main hydrostratigraphic units and several sub units (Omer *et al.* 1981). The main units are the upper aquifer, lower aquifer, and intermediate aquifer. The upper boundary of Kuwait Group lies 10 to 30 meters (30-100 feet) below the ground surface. The aquifer varies in thickness from 5 to 200 meters and is composed of sands, gravel, clays and limestone. Figure 4-8 presents Kuwait Group potentiometric levels recorded in Kuwait. The potentiometric levels are around 6-12 meters (20-40 feet) in the north eastern part of the country, which gradually increases towards the western and southwestern parts of the country with levels reaching 94 meters (308 feet) near the border area. Infiltration of water from surrounding countries and upward flow of Dammam Formation are the two main sources of groundwater in the main Kuwait Group.

Dammam Formation, the most widespread aquifer in Kuwait containing mobile groundwater, occurs at depths ranging from the surface in the southwestern parts to 200-meters in the northeastern parts of the country (Milton 1985). Dammam is exposed along the Al-Ahmedh Ridge, about 40-km south of Kuwait City. The average thickness for the aquifer ranges from 123-meters (403-feet) in the southwestern parts of Kuwait to about 300-meters (1,000-feet) in the northern section of the country (Al-Jawadi *et al.* 1993, Younis 1994). Agriculture districts depend heavily on water well fields pumped directly

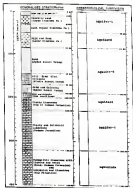


Figure 4-4 Hydrological Subdivisions of the Aquifer System in Kuwait (after Mubarek et al. 1994)



Figure 4.7 Hydrogeological cross-section of the Apurimac region in the Apurimac System (adapted from Al-Wadi 1995)



Figure 4-6 Keweenaw Group Metamorphic Limits at Keweenaw
(adapted from AG-Map, 1970)

from the Dannebak Formation. Benbow (1964) reported that 13,000 billion gallons of freshwater water is stored in the Dannebak Formation. The top of the Dannebak Formation is composed of 5-10 meters of semi-impermeable shales and siltites (Yousif 1984). Al-Karaki et al. (1993) reported that the Dannebak Formation has a low effective primary porosity and its water-supply characteristics depend on secondary porosity that may be caused by fracturing, faulting, karstification, and dolomitization. The measured permeability for the Dannebak Aquifer is about 30 square meter per day (Abdullah and Mollah 1997).

Geology and Topography of Soil Layers

Karasi is a small country located at the northeast edge of the Arabian Peninsula and bordered by the Arabian Gulf at the east. The surface topography of Karasi is generally flat or gently undulating desert plain (sandy and gravelly), sloping toward the sea, with low isolated hills, escarpments, and shallow closed depressions. Furthermore, the Karasi surface elevation tends to decrease towards the eastern part where the landscape rises from sea level to a maximum point of about 174 meters (569-feet) in the southwestern corner of the country. The mainland of Karasi slopes gently towards the sea at an average gradient of 1 to 100 (Jassidy 1984). A topographic map of Karasi is shown in Figure 4-5.

The surface of Karasi is formed of sedimentary rocks and sediments ranging from early Miocene to Karasi (Figh 1964). Sedimentary succession in Karasi can be divided to two main groups: Karasi Group, and Hama Group. Karasi Group lies at the surface of the eastern area of Karasi except where marked by recent deposits (Figure 4-11). Karasi is Sub-recent Formation is classified late-olocene sands, residual deposits,

playa deposits, and coastal deposits that consist of desert plain deposit and fluvial deposit. The formation contains fine to coarse-grained sands and plenty of gravel and boulders in depressions and cemented sand with silt, clay, and precipitated calcium and magnesium carbonate along the east-coast of Kuwait City (Omer et al. 1981, Fagh et al. 1988).

Kuwait Group can be divided to three formations, Dibdibba, Lower Fars, and Qhar. The Dibdibba Formation, mainly located in the northern parts of Kuwait, consists of cross-bedded sands and gravel, sandy clays, sandstone, and sands partially cemented by salt or gypsum. The Lower Fars consists of five subformations, sandstone, calcareous pebbly sandstone, sandy limestone, and sandy sandstone (Omer et al. 1981, Younis 1986). According to the Ministry of Oil, the thickness of the Fars Formation ranges from 81-m (266-ft) in the west to 149-m (489-ft) in the north. The Qhar Formation lies below the Lower Fars and overlies the Dammam Limestone. The Qhar Formation is composed mainly of coarse sands and gravel with some sandy limestones and sandy clay and calcareous muds (Younis 1986, Omer et al. 1981). According to Omer et al. (1981), the thickness of the Qhar Formation increases from southwest direction towards the north and northeast direction reaching a maximum thickness of 143 meter (469-feet).

The most important formation in the Fars Group is Dammam. The Dammam Formation consists of 12 to 274 meters (390-900 feet) of soft, porous, chalky limestone and hard crystalline dolomitic limestone with shells of the fauna. In most of Kuwait State is underlies the Qhar Formation of the Kuwait Group. At the northern, Dammam limestone is of economic importance where hard rocks are rare in Kuwait. The limestone is used to manufacture local building stone.



Figure 4-5 Topographic Map of Korea (adapted from Osner 1981)

AGE	FORMATION	FORMA-TION	LITHOLOGY	DESCRIPTION
NEO-GENE		RECENT AND RECENT		10 PLANS REPORTED ON GRAVEL BEDS SAND, SAND, SILT AND CLAY
PLIO-GENE	SERRA SERRA	ODONDA		SAND-GRANITE, GRAVEL, SANDSTONE, SAND, SANDY BEDS, SANDSTONE
MIocene FLUORIDE		LOWE'S FAIR ALL-AT TOP		SANDY BEDS SANDSTONE SANDSTONE, SANDY SANDSTONE, SANDSTONE SANDY Limestone
Eocene- CENE			WITLA	
		CHAS		
EO-GENE	NEOLA	QUARLAN		CHERT CHAS, SANDSTONE Limestone AND SANDSTONE

Figure 4-10 Stratigraphic Subdivisions of the Aquila System in Kuwait
(adapted from AJ Rana and Masood, 1990)

Soil Types

The soil profile in Kuwait typically consists of a surface layer of windblown sand and underlain by a silty fine to medium sand layer, usually cemented, known locally as Gachs. Below the Gachs deposit, limestone bedrock is encountered. Kuwait's soil characteristics have been affected by numerous environmental factors briefly listed as follows:

1. High temperatures in the summer which reduce the moisture content to low values
2. Evaporation predominates over rainfall that leads to concentration of soluble materials near the surface due to the upward movement of soil water by capillary action
3. The hot and dry winds in the desert, containing no remaining moisture for precipitation, resulting in the formation of fine rounded sand grains

The three main soil deposits in Kuwait are surface windblown sands, coastal salt bearing soils (Bahls), and cemented calcareous sands (Gachs). A brief review of the geotechnical properties, composition and variability of these soils will be presented briefly in the following sections. A detailed account of the properties and behavior of soil deposits in Kuwait was presented by Ismail (1988, 1993, 1995), Ismail et al. (1990, 1992, 1998), Al-Sulaiman et al. (1982), and Banerji and Abu-Idris (1992).

Surface Windblown Sands

The surface windblown sands consist of predominantly fine sand with some medium sand, no gravel, and traces of fines. The percent of fines passing the No. 200 US sieve ranges between 3.2 and 15.3% and on average it forms 10% of the total

composition. The Unified Soil Classification classifies Kuwait surface sands as poorly graded sand to silty sand (SP-sM). Furthermore, the coefficient of permeability ranges between 10^{-2} to 10^{-4} cm/sec. The specific gravity varies between 2.67 and 2.72 and it has low natural moisture that ranges from 1 to 2%. Its thickness ranges from 0 to 7 meters below the ground surface. Surface deposits are suitable as a backfill material and can support light to moderate pressures. On the other hand, it is sensitive to saturation or ground wetting. Consequently, collapse upon wetting is possible if not well compacted.

Coastal Salt Bearing Soils (Sabkha)

Salt bearing soils called locally as Sabkha are coastal flat areas that extend above the high tide level and are covered by evaporate-rich shallow sediments (Khalaf et al. 1944). It has a large content of salts in the form of carbonates and gypsum. Sabkha consists of heavy gyp-sediment fine sandy silt with little clay and occurs along the coastline of Kuwait. The Unified Soil Classification designated Sabkha as ML. Moreover, The specific gravity ranges between 2.3 to 2.5 and the moisture content varies up to 35%. Percent of fines passing the No. 200 U.S. sieve is more than 50%. Furthermore, its coefficient of permeability on average was recorded as 3×10^{-3} cm/sec.

Sabkha has large concentrations of sulfates which exceed 60% of the soil composition at ground surface and decreases sharply with depth. These soils are not suitable for backfilling or direct contact for foundation support. The very low bearing capacity of this soil, and its corrosive reaction with concrete and steel makes it unsuitable for foundation placement without soil improvement. Also, due to sulfate concentrations, Sabkha is subject to volume change. Sulfates consist of either gypsum or anhydrite. At high temperatures, volume change due to dehydration of gypsum is anhydrite followed

by dehydration in the presence of water. Finally, leaching soluble salt leads to increase permeability and compressibility and reduce shear strength.

Coastal Calcareous Sands (Qasbi)

The Qasbi deposit is the local name for the calcareous accumulation which is characteristic in arid and semiarid climates. In these areas, with semiarid climate conditions, generally in calcareous, to retain the calcareous formation due to evaporation. All stages of calcareous accumulation are observed in Kuwait such as primary granular calcareous material, or sandy deposit with isolated calcareous pods, or hard calcareous deposits. The depth of the Qasbi layer, ranging from less than 0.5 to 3 meters from ground surface, varies from one location to another, but generally, the depth increases towards the Arabian Gulf.

Qasbi is composed of an unconsolidated calcareous material, which contains variable amounts of material, in order of abundance: like Quartz, K-Feldspar, Gypsum, Fossils and Shell Fragments, Plagioclase, and Hemihexite and Pyroxene. Calcareous material percentages in Qasbi have an average of 80-90%. Quartz material ranges between 17.6 and 41% with an average of 32% and K-Feldspar material have an average of 3.3%. Gypsum material ranges between 8.8 and 14% and other materials like Fossils and Shell Fragment, Plagioclase, and Hemihexite and Pyroxene have average of 1.6%, 8.1%, 8.08% respectively. Near the seashore, the Qasbi is composed mainly of quartz, ranging in size from 0.05 mm up to 1-mm, feldspar and some macrofossils and corals. All these components are cemented by fine-grained matrix of calcite (less than 0.05 mm). In other locations, the Qasbi is composed of variable percentages of feldspar, feldspar potholes and nodules, plagioclase, quartz, and ferromagnesian minerals such as clinochlore and

amphiboles. In some rare cases, gypsum replaces the calcite cement. The calcareous material is usually very fine in grain size and it usually replaces the quartz and other minerals in the rock. Most of the clay size particles are not clay minerals comparing to the clay fraction. The clay mineral content is only 12%, on average, of the clay size particles and is composed of chlorite and illite minerals. The remaining clay size particles include carbonates, oxides and minerals in very fine form. Soil classification by Unified system, AASHTO system, and Folk (1974) are shown in Table 4-1 and Gatch physical properties are presented in Table 4-2.

The major elements for the Gatch, in terms of abundance, are SiO_2 , CaO , MgO , Al_2O_3 , Fe_2O_3 , K_2O , and TiO_2 + SnO_2 , which reflects the amount of quartz, ranges between 34.87 and 51.35%. CaO and MgO average percentages in Gatch are 12.9 and 3.7 respectively. CaO and MgO account for the calcareous material, present either as calcite or dolomite. The amount of CaO and MgO increases as SiO_2 decreases. Al_2O_3 content, ranging from 6.11 to 8.36%, is attributed to the presence of feldspar and some ferromagnesian minerals. Fe_2O_3 content which average percentage in Gatch is 6.8 is related to pyroxenes, amphiboles, and iron oxide minerals such as magnetite. K_2O content averaging 0.45%, is attributed to potash feldspar and some clay minerals.

According to Savely (1984 and Alvo-Ed, and Ismail et al. 1984), sandy calcareous soils are highly susceptible to water expansion and are particularly affected by water flowing through them. Several environmental factors contribute to the swelling problem and include the very low moisture content of the near surface soils and the high suction potential, loss of cementation bonds, and the presence of clay and silt near cohesion

(Issawi et al. 1996). Table 4-3 presents a comparison between properties of local soils in Kuwait.

Table 4-3 Grain Size Classification by the different systems (adapted from Issawi 1998, Al-Jalham et al. 1982).

Classification System	Description
Unified Soil Classification	<ul style="list-style-type: none"> Most samples are classified as SM (silty sands, sand-silt mixtures), and/or SC (clayey sands, sand-clay mixtures)
AASHTO Classification	<ul style="list-style-type: none"> Most samples are classified as A-3-4 (silty or clayey gravel and sand, % passing No. 200 = 15 mm, U₁ = 40 mm, P₁ = 10 mm.)
Grain Size (Folk 1974)	<ul style="list-style-type: none"> Grain size size (d_{50}) ranges from coarse sand (1-4.75 mm) to fine sand (0.25-0.125 mm), 45% of the samples are Coarse sand, 39% Medium sand (0.5-0.25 mm), and 16% fine sand Inclusive graphic standard deviation (σ) ranges from moderately well sorted (0.5-0.31 ϕ) to very poorly sorted (1.0-0.8 ϕ). 67% of the samples are poorly sorted, 36% very poorly sorted, and 13% very well to well sorted Inclusive graphic skewness (S_k) ranges from strongly fine-skewed (ϕ 0-0.5) to strongly coarse skewed (-0.3 to -1 ϕ). 32% of the samples are fine skewed, 32% are symmetrical, 27% coarse skewed, 6.3% strongly fine skewed, and 6.5% strongly coarse skewed. Congluic kurtosis (K_u) ranges from very platykurtic (under 0-07) to very leptokurtic (0.3-0.5), 42% of the samples are mesokurtic, 26% leptokurtic (the central portion is better sorted than the tails), 22.6% platykurtic (the tail is better sorted than the central portion), 6.4% very leptokurtic, and 7% very platykurtic

Table 4-3 Physical Properties of Cemented Sand in Kuantan/Johor (area) 1998. AG-Rafiana et al. 1997)

Parameters	Range	Mean
Sand (%)	41.6-82.3	62.8
Fine (Silts&Clay) (%)	17.3-58.4	37.1
Clay (>0.002 mm) (%)	0.5-38.8	16.3
Clay/Fine (%)	38.7-64.4	49.8
Plasticity Index (%)	4-17	10.8
Liquid Limit (%)	28-68	38.1
Plastic Limit (%)	18-32	23.4
Shrinkage Limit (%)	17-19.8	18
Specific Gravity	2.63-2.74	2.68
Voidity	0.46-0.78	0.63
Free Swell (%)	0.3-29.3	9.64
Permeability (cm/sec)	$10^{-4} - 10^{-2}$	10^{-3}
Max. Dry Unit Weight (kN/m^3)	18.24-28.7	19.42
Optimum Moisture Content (%)	8-12	10.8

Table 4-3. Comparison between Properties of Local Sands in Kuwait (after Ismail 1998).

Proprietary Soil Type	Windblown Sand	Coastal Sand	Compacted Sand
Description	Loose to medium dense fine to medium sand, with little silt	Loose Duple-dense fine sandy silt with little clay	Very dense, silty sand or clay sand with cemented bands or layers
Classification	SP-1SM	ML	SM, SP-1SM, SC
Strength Parameters	$C' = 0$, $\phi' = 30^{\circ}-35^{\circ}$	$C' = 0$, $\phi' = 25^{\circ}-30^{\circ}$, for cemented sand $C' = 10-20$ kPa, $\phi' = 30^{\circ}-35^{\circ}$	$C' = 10-20$ kPa, $\phi' = 30^{\circ}-40^{\circ}$
Permeability (cm/s)	$1 \times 10^{-3} - 1 \times 10^{-1}$	$1 \times 10^{-3} - 1 \times 10^{-1}$	$10^{-4} - 10^{-2}$
Percent of Fines (%)	5-15	<50	10-20
Composition of Fines	Calcareous with ~ 50% carbonate & sulfates, 40-45% silica	Mainly sulfates and carbonates	~30% carbonate, 40-45% silica
Disadvantages	Possible collapse upon wetting, especially if not well compacted	<ul style="list-style-type: none"> - Very weak, not good for foundation support - Loose strength upon wetting and aggressive leached carbonate and steel - Salt (crystallizing) effect and properties and subject to volume change 	<ul style="list-style-type: none"> - Possible swelling if it contains >20% of fines especially if used as compacted landfill - Poor bearing capacity with depth
Advantages	<ul style="list-style-type: none"> - Reliable as landfill material - Can support light to moderate pressures 	<ul style="list-style-type: none"> - None 	<ul style="list-style-type: none"> - Compaction very dense deposit - Can support large pressures - Can be used as landfill - Small settlement developed

CHAPTER 5 TWO-STAGE BOREHOLE PERMEABILITY TEST

Basic Concept

The Two-Stage Borehole (TSB) is a falling-head infiltration test conducted in a cased borehole, typically 4-inches in diameter, to control the geometry of the infiltration zone (Frostwell 1997). The annular space between the casing and the hole is sealed with grout. The test consists of two stages that vary in infiltration geometry to reveal the field measurement of limiting hydraulic conductivities. These limiting hydraulic conductivity values are the maximum possible for the vertical direction (K_1), and the minimum possible for the horizontal direction (K_2). Combining the infiltration results from the first and second stages allows for the estimation of both the actual vertical and horizontal hydraulic conductivities in the test zone.

In the first stage, the test is performed using a flat bottom flush with the base of the casing for the maximum vertical permeability (Figure 5-1). This stage of the test is completed when K_1 reaches some value that is a steady state. The second stage involves uncasing the casing, and deepening the hole (5 to 8 inches) below the bottom of the casing by either pulling a thin-walled tube or auguring a deeper hole into the soil and removing the removed soil by wire brush (Domen 1996). The apparatus is refilled and a falling-head test performed again until it achieves steady state. At this stage, the minimum hydraulic conductivity in the horizontal direction is measured (Figure 5-2).

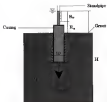


Figure 5-1 Step 1 of the Two-Stage Borehole Field Permeability Test

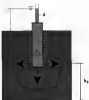


Figure 5-12 Stage II of the Two-Stage Borst's Field Penetration Test

Coastal (1989) stated the major assumptions for TSI method. These assumptions are listed as follows:

1. The soil tested is homogeneous and uniformly soaked with water
2. Soil is not saturated across the surface
3. The pore-water pressure is zero at the base of the casing (stage I) or the center of the uncased section (stage II)
4. Soil surface effects are negligible and boundaries are at infinity
5. During the test, the soil undergoes no volume change
6. Haverly's (1945) equations are valid

Significance and Use

The TSI is commonly used for measuring hydraulic conductivity of compacted clay liners and covers used at waste disposal facilities, for canal and reservoir liners, for seepage blankets, and for amended soil liners such as those used for retention ponds or storage tanks (ASTM Z23092). According to Boothwell (1952) and Sharma and Sangsri (1994), the test has been successful in evaluating both compacted and natural materials with permeabilities as low as 1×10^{-10} cm/sec.

The soil being tested must have sufficient volume to stand open during excavation of borehole and have a uniform permeability and pore space distribution (ASTM Z23092). 10-cm (4-inch) inside diameter (ID) casing has been used in virtually all of the known tests. To avoid existing boundary constraints assumed in the equations, the thickness of the soil tested must be 41 cm (24-inches) or 41.3 cm (12-inches) if the barrier is not underlain by drainage blanket or by a material far less permeable than the soil tested (ASTM Z23092). Also, the minimum casing embedment below ground

surface must be 25.4 cm (1.0 inches) to prevent uplift and minimize hydraulic fracturing (Bourwell 1992). Furthermore, the minimum horizontal distance between tests or tests to free surface are 3-m (120 inches).

A typical test program consists of five TSB's and a temperature effect group (TEG). The TEG is used as a control device to determine changes in head caused by variations in temperature and barometric pressure (Denson et al. 1987). TEG is a TSB that has a sealed bottom to prevent flow and is fitted with digital thermometer for tracking temperature changes in the wellbore water. The significance of TEG is the fact that it can measure hydraulic conductivity in vertical and horizontal direction. Most of the field permeability tests do not have this advantage. Also, TSB is easy to install and can be used at great depths and on slopes (Denson 1989).

Apparatus and Field Procedure

TSB field procedure is taken from long-established US Bureau of Reclamation methods, Dash bottom borehole test (D-18) and borehole packer test (D-19) (Bourwell 1992). American Society of Testing Materials (ASTM) has recently approved a standard method for field measurement of hydraulic conductivity limits of porous materials using TSB (ASTM E2190C). This method is utilized for compacted fills or natural deposits, above or below the water table, that has a mean hydraulic conductivity less than or equal to 1×10^{-4} cm/sec.

The methodology for installing and conducting the TSD tests at Kansas followed the procedure recommended by ASTM D2592 with certain alterations to serve the purpose of the research. These alterations are described in the following:

1. Fine sand was used as a permeable layer to underlie the compacted fill
2. Paraflex tentacle is used to seal casing due to its availability
3. Compacted cement sand pads required shorter period of curing times than the suggested elapsed times in Table 3 of ASTM D2592. Nevertheless, testing on the deep cemented sand areas followed the recommended curing times
4. Kram High Sacks were used to protect the soil at the bottom of the casing from water disturbance and to prevent collapse of the Stage 2 cover. The sacks are described as flexible thin puncturable nylon that are sealed at the bottom, porous side-walls and bottom, and filled with pea gravel. The water diameter is 9.5-cm (3.7-inch) and the length is 45-cm (18-inch). Due to its stretchability the sacks are removed easily by hand
5. The temperature system consists of an Omega pocket digital thermometer accompanied with PTC coated tip thermocouple. The Omega 18MC216-KF hand held units are digital thermometers that can be used to measure temperature from -58 to 300°F using integral thermocouple probe, or from 0 to 1500°F using tip type K thermocouple with subminiature connector. The resolution of this unit is 1°F , 0.2°C with 32 to 122°F (0 to 50°C) operating ambient temperature
6. 15x2.25 meters of clear plastic sheeting, nominal thickness of 6.15-mm, was used as a protective on the compacted layer

Estimate Corrections to Calculations

TEO permeability computation are based on the Hoerner (1949) equations adapted for various bottom/boundary conditions by the three dimensional Image Potential Technique (Cuthrell and Jager 1959). The data from each reading is converted into an apparent permeability, termed (K1) for Stage 1 and (K2) for Stage 2. The equations for both Stage 1 and Stage 2 follow the generic falling-head equation

$$K = \frac{G R_{wT}}{(t_2 - t_1)} \ln \frac{H_0}{H_1} \quad (1)$$

where,

K = Permeability

H_0 = Effective head at the beginning of time increment (cm)

H_1 = Corrected effective head at end of time increment (cm), $= H_0 - i$

t_1 = Time at the beginning of increment (s)

t_2 = Time at the end of increment (s)

G = Geometric constant, depending on test geometry

R_{wT} = Kinematic viscosity correction factor to water at 20°C (68°F)

In both stages, the head is equal to the distance from top of water in standpipe to the top of the underlying stream or groundwater, whichever shallower. If the water level is extremely deep, the depth is limited to 20 test chambers below the casing. The term H_1 is the measured head (H_0) less any increase in TEO standpipe level during the time increment (s). If the TEO standpipe water level goes up between readings, then (s) is positive and $H_1 < H_0$. On the contrary, if the TEO drops between readings then (s) is

negative and $(\Delta T)^{-1} = B_2$. The B_2 factor corresponds to the average fluid temperature during a time increment, relative to water at 20°C (68°F). The factor B_2 is given for water at temperatures between freezing and 120°F (3 to 48°C) in ASTM D584. The geometrical factors are as follows:

Stage I

$$Q_{L_{\text{ex}}} = (\pi d^2 / 4) h_m (\Delta T_1) + \pi (D_1 / 4) h_m b_1 \quad (3)$$

Stage II

$$Q_{L_{\text{ex}}} = (\pi d^2 / 4) h_m F \left[\frac{L_1 \left[w(m, L_1 + T, \alpha) \right] + \pi h_m \left[w(m, L_1, 2b_1) \right]}{1 + F \left[L_1 \left[w(m, L_1, \alpha) \right] / w(m, L_1 + T, \alpha) \right]} \right] \quad (4)$$

where

L_1 = Length of Stage II extension below bottom of casing (mm)

F = $1 - 0.5625 \exp(-1.568L_1/D_1)$

$$w(m, L_1, \alpha) = \left[mL_1 / D_1 + \sqrt{1 + (mL_1 / D_1)^2} \right]$$

$$w(m, L_1, 2b_1) = \frac{4mb_1 / D_1 + mL_1 / D_1 + \sqrt{1 + (4mb_1 / D_1 + mL_1 / D_1)^2}}{4mb_1 / D_1 + mL_1 / D_1 + \sqrt{1 + (4mb_1 / D_1 + mL_1 / D_1)^2}}$$

$$w(m, L_1 + T, \alpha) = \left[mL_1 / (D_1 + T) + \sqrt{1 + (mL_1 / (D_1 + T))^2} \right]$$

F = Bore ratio, $= K_2 / K_1$

K_1 = Permeability of cement zone

T = Thickness of cement zone (3.6-in)

d = Inside diameter for Stage I

D_1 = Diameter of Stage I (mm)

D_2 = Diameter of Stage II extension (mm)

- b_1 = Thickness of sealed well below base of casing (cm)
 b_2 = Distance from center of Stage 2 screen to top of waterfiring screen of ground water (cm)
 n = -1 for permeable bottom boundary
 0 for infinite depth to bottom boundary
 +1 for impervious bottom boundary
 m = Parameter that defines the degree of anisotropy, $= \sqrt{k_{xy}/k_z}$

The limiting vertical hydraulic conductivity ($K1$) for each time increment is calculated by equations (1), assuming $(m=0)$ in equation (2) to yield (3). Then the arithmetic time weighted average $K1$ of the $K1$ values for Stage 1 during the quasi-steady period is calculated by the following equation:

$$K1 = \sum K1_i \Delta t_i = t_1/k / \sum t_1/k \quad (8)$$

where:

- $K1$ = Arithmetic time weighted average permeability (cm/sec)
 t = Specified time increment

Similarly, the time-weighted average $K2$ of the $K2$ values of Stage 2 for its quasi-steady state flow periods is calculated by using equation (4):

Assuming the soil tested is homogeneous, then the vertical permeability must be the same in both stages. Hence, a unique equation that is a function of $(m, L/D, \text{ and } F)$ is established:

$$K2/K1 = (S1_{ss}/Q1)^2 (Q2/Q1_{ss}) \quad (9)$$

where

$$\begin{aligned} (QI_a/QI) &= (1+\alpha(D_1/4\pi h_1)) \left[(1+\alpha(D_1/4\pi h_1)) \right] \\ (QI/QI_a) &= \frac{\pi^2 \left[\alpha \left(\frac{1}{2} \sqrt{\frac{1}{2} \frac{h_1}{\alpha}} \right) + \alpha \left(\frac{1}{2} \sqrt{\frac{1}{2} \frac{h_1}{\alpha}} \right) \right]}{h_1 \left[\alpha \left(\frac{1}{2} \sqrt{\frac{1}{2} \frac{h_1}{\alpha}} \right) + \alpha \left(\frac{1}{2} \sqrt{\frac{1}{2} \frac{h_1}{\alpha}} \right) \right] + \pi^2 \left[\alpha \left(\frac{1}{2} \sqrt{\frac{1}{2} \frac{h_1}{\alpha}} \right) + \alpha \left(\frac{1}{2} \sqrt{\frac{1}{2} \frac{h_1}{\alpha}} \right) \right]} \end{aligned}$$

Since the ratio of (K_2/K_1) , and the geometric terms are known, then equation (7) is solved by taking a suitable value of (P) and determining by trial and error the value of (m) . The normal ranges of (P) values are listed in Table 3-1. A value of (P) equals to 1 indicates that no seepage has occurred. The actual values for the horizontal and vertical permeability are calculated by the following equations:

$$k_a = (K_1/QI_a)/G \quad (8)$$

$$k_v = m^2 k_a \quad (9)$$

Table 3-1. The Normal Range of (P) Factor (adapted from Dwyer 1992)

(K_2/K_1)	P
> 1.1	1
0.9-1.1	1.2
0.8-0.9	2.1
0.7-0.8	5.0
0.6-0.7	10.0
0.5-0.6	15.0
0-0.5	"

* Use Range 1 only

Case Studies of Success and Failure

The TBS method was first used in 1963 while the preliminary procedure was published by Deschell and Senack in 1966. According to Deschell and Tan (1992), the test was successful at wells ranging from CH-OH (Lapack Index > 30 and clay content 75%+) to BC/OO (Lapack Index < 30 , and clay content 12%) and natural seals to depths of 6 meters and in some oilfields made to depths of 18 meters. Furthermore, the degree of compaction to the elevated test pads was between 90 and 100% of Standard Proctor test, at water contents ranged between 2% below optimum and 4% above of optimum.

The purpose of these case studies is to illustrate the common practice of using TBS in the field, and the adequacy of results of hydraulic conductivities measured by this method.

Case Study 1 (C)

TBSs were used, along with sealed double-ring calipers (SCCs) and laboratory testing on large blocks and small samples, to assess the hydraulic conductivity of four test pads in October 1962 (Rousseau et al. 1997). This assessment was required prior to the construction of an artificial cap-lin closure of a 14-hectare surface impoundment of a chemical plant in Pompey, Texas. Four different contractors constructed the four test pads to the same specification with low plasticity clay.

The four pads (A, B, C, and D) were 40-m long, 8.9-m thick, and 1.8-m wide except for test pad B which was 22-m wide to accommodate water construction equipment. A fine-draining sand, at least 8.13-m thick was placed under each test to provided a known boundary condition for hydraulic conductivity comparisons. Each pad

was compacted at water content ranging between optimum and 7% wet of optimum and to a maximum dry unit weight of at least 95% of standard Proctor.

Five TSDs were installed in each test pad for measuring hydraulic conductivity and an additional one, sealed from bottom, to account for temperature variations and barometric pressure. The test pads were subjected to environmental distress during one summer and one winter, even though a layer of polyethylene and a layer of sand protected the test pads. TSDs had an inside diameter of 8.1-in (206mm) and were set to depths between 8-10 and 1-46 meters below the ground surface. The values of the natural hydraulic conductivity measured by TSD tests and the other different test methods are summarized in Table 3-3.

The hydraulic conductivities measured by TSDs were comparable those measured by the long-term Sealed Double-Ring Infiltrometers (SDRIs). On average, TSDs hydraulic conductivities were about twice the hydraulic conductivities of SDRIs. Apparently, all results were within the required field hydraulic conductivity ($<1 \times 10^{-7}$ m/sec) that was specified by the project consultant.

Case Study (2)

A study was performed on clay liners at a waste disposal site in Louisiana to compare laboratory and field hydraulic conductivity, and to establish a statistical relationship between laboratory hydraulic conductivity tests and clay liner permeation rates during construction (Johnson et al. 1998). Two prototype clay liners, A and B, were built with construction specifications similar to the actual cell liner construction. TSDs, SDRIs, and Pan Lysimeters were used to measure field hydraulic conductivity. Flexible Wall and Fixed-Ring laboratory tests were used to measure laboratory conductivity

Table 5-1: Increased Hydraulic Conductivity by the Different Field and Lab Tests (Adapted from Benson et al. 1997)

Test Field	Sampling Tube	Block Specimens	TSB	SCB2 (1), (Dec. 1997)	SCB2 (2), (Aug. 1998)
	(m/s)	(m/s)	(m/s)	(m/s)	(m/s)
A	2.2×10^{-10}	4.8×10^{-10}	3.1×10^{-10}	3.6×10^{-10}	3.2×10^{-10}
B	4.8×10^{-10}	7.7×10^{-10}	5.2×10^{-10}	1.5×10^{-9}	6.8×10^{-10}
C	3.8×10^{-10}	3.1×10^{-10}	2.3×10^{-10}	2.2×10^{-10}	2.6×10^{-10}
D	2.1×10^{-10}	3.3×10^{-10}	1.1×10^{-10}	1.3×10^{-10}	1.1×10^{-10}

values.

Layer A was constructed in 1986 using four 15.24-cm (6-inch) thick compacted lifts to an approximate size of 13 x 38 meters (20 x 100 feet). During construction, the clay moisture content ranged between 8 to 6 percent wet of optimum, and the average density was 93.8% of the standard Proctor maximum density. Six TSBs, three 3-cm (3/8-inch) diameter and three 10-cm (4-inch) diameters, were built and set on the 68-cm (2-foot) clay liner that lays over a compacted 0.3-m (1-ft) thick soil base where hydraulics (pneumatic collectors) and outlet piping were installed. The first stage of each test achieved a steady-state condition in 6 to 7 days while the second stage steady-state condition was achieved in 7 to 8 days.

It was interesting to note that T&Es hydraulic conductivity results were almost similar to the lab and lysimeter tests results. The mean value for the vertical hydraulic conductivity for T&Es was 6×10^{-7} cm/sec while the lysimeter, which was monitored for a period of four years (1986-1990), yielded hydraulic conductivity values ranging from 5 to 4×10^{-7} cm/sec while the results from laboratory tests on undisturbed samples averaged 3×10^{-7} cm/sec.

CHAPTER 4 LAB TESTS

Sampling and Testing Program

It is the purpose of this study to investigate the permeability characteristics of stabilized sands in Kuwait and check their applicability as liner or cover systems for solid waste containment units. To achieve this goal, bulk disturbed samples of the unknown quartzite sand, known locally as 'quish', were collected from three areas, namely South Sura (SS), Jaber (JB), and Al-Qurain (QK) which are located within the urban limits of Kuwait City (Figure 4-1). In order to obtain parameters that can be used for comparison with the standard characteristics of natural soils, the testing program was repeated on a soil mixture obtained by adding 5% bentonite (by weight of dry soil) to South Sura soil and an infiltration test was conducted on Jaber samples. Furthermore, the Al-Qurain sample was permeated by chemical solution and water, and compared at different water contents. Soil samples were tested in the laboratory in accordance with ASTM (2005).

The testing program comprised first of determining the physical and index properties needed for soil classification. Comparison tests were performed on soil samples using the ASTM G1 937 procedure for South Sura and Jaber soils, and the AASHTO T144-78 procedure for Al-Qurain soil to obtain the optimum moisture content and the maximum dry density values for use in permeability determination of the stabilized soils (Figures 4-2 to 4-5). Later, samples were compared to the same conditions that were achieved in



Figure 4b-3 Locations of Soil Samples Collected for Lead Tests



Figure 4-2 Moisture and Dry Density Relationship of Soil Collected from South Sams

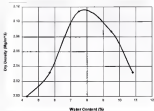


Figure 4-3. Moisture and Dry Density Relationship of Soil Collected from South River Mixed with 5% Bentonite

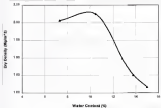


Figure 6-4 Moisture and Dry Density Relationship of Soil Collected from Idara

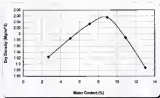


Figure 4-5. Moisture and Dry Density Relationship of Soil Collected from Al-Qayman

the field and Consolidated Undrained Triaxial tests (CU) accompanied with Falling Head Permeability tests were conducted on these samples to measure permeability. The testing programs for the soils are given in Tables 6-1 and 6-2.

Soil Basic Properties

Soil analyses, using the wet method, were performed on natural soils, and Atterberg limit tests were conducted on the portions of these soils passing the sieve No. 40 in order to obtain the necessary information to determine their basic engineering properties and classification. The grain size distribution curves are shown in Figure 6-6. The figure shows that the content of fines (percentages passing No. 200 U.S. sieve) for the two sands (SS & SL) are almost the same, about 11%, however, the AL-Qusais sample has a higher value than the others, about 13%. South Sana and Jibsa sands can be described as fine to medium, with a uniformity coefficient of about 6.5, an effective grain size of 0.08 mm, and median grain size of 0.16-0.29 mm. The specific gravity of the sands was determined as 2.66 and 2.67 respectively, which are typical for sands in Kuwait (Jumari et al., 1990). Al-Qusais sand is described as silty sand with a median grain size of 0.27-mm.

The natural moisture content of the soils ranged between 1.9 and 5.3% indicating a general moisture deficiency. Visually, the samples showed some aggregation of the particles, or some particles are sticking to the large grains. Although it was not determined quantitatively for this study, the matrix is largely carbonates (Jumari et al., 1987). The results of Atterberg limit tests on fractions passing No. 40 were indicated that plastic and liquid limits were not determinable, and therefore, these were recorded as non-plastic (NP) according to ASTM D4318.

Table 4-1 Consolidated Unconsolidated Triaxial Test (CU) Testing Program

Soil	Area	Index Properties	Poreability Determination Program			
			Series	Breakdown (%)	Cycles of drying & wetting	No. of test
SL-1	Riser	Stress Anisotropy Index	K-1	0	0	1
			K-2	0	3	1
			K-3	0	6	1
SL-2	South Barr	Specific gravity Compaction	K-4	0	1	1
			K-5	3	0	1
			K-6	3	3	1

Table 4-2 Falling Head Test Testing Program

Soil	Area	Properties	Permeability Determination Program				
			Series	Water Content (%)	Dry Density (Mg/m ³)	Leached by Chemical Solution	No. of test
SL-1	Ad-Quartz	Stress water tests	K-7	4	1.90	Yes	1
			K-8	4	1.90	No	1
		Compaction	K-9	9	2.00	Yes	1
			K-10	9	2.00	No	1
		Chemical Composition	K-11	11.6	1.90	Yes	1
			K-12	11.6	1.90	No	1

Al-Sabawi et al. (1998) conducted compaction tests on 17 subgrade soils from different areas in Kuwait City and its Suburbs. Out of the 17 samples, 16 were found to be NP, and therefore, the present data are within the limits. 5%, 35, and 6% sands are respectively classified as SW-SM (poorly graded sand), SP-SM (poorly graded sand with silt), and SM (silt) according to the Unified Soil Classification System. South Sana'a soil is composed of 13% gravel, 42.9% fine sand, and 44.1% silt and clay. Jabraa soil is composed of 5.2% gravel, 52.8% fine sand, and 41.9% silt and clay. Likewise, Al-Qasbi soil is composed of 3% gravel, 50% coarse sand, 30% fine sand, and 17% silt and clay. Complete basic properties of the soils are given in Table 6-3.

The results of the compaction tests are included in Table 6-4. Table 6-4 shows that the optimum moisture content for South Sana'a, Jabraa, and Al-Qasbi are 8.7%, 9.7%, and 8.8% respectively, and the corresponding maximum dry density values are 2.68, 2.65, and 2.66 Mg/m³ respectively. Moreover, the optimum moisture content for the South Sana'a soil mixed with 5% bentonite is 7.8%, and the corresponding maximum dry density value is 2.13 Mg/m³. The test result indicates that the addition of 5% bentonite slightly changed the characteristics of the sand material.

Determination of Permeability

Consolidated Undrained Triaxial Test

Test arrangement. The laboratory determination of permeability was carried out first using constant hydraulic gradient in a standard triaxial cell as shown in Figures 6-7. Constant transducers were used for measuring test data while a fully computerized system comprised of an auto-data acquisition unit and a software package, supplied by Engineering Laboratory Testing Equipment (ELTE), England, was used to monitor the

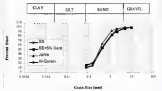


Figure 4-6 Grain-Size Distribution for the Soil Samples

Table 4-3 Summary of Properties of the South Seas and Johns Sands

Sample ID	Material SOC	ASTM D421						Sp. gr.	Atterberg limits		Unified soil classification
		Gravel (%)	Coarse sand (%)	Medium sand (%)	Fine sand (%)	Silt & clay (%)	Moist. wt. (%)		Liquid limit (%)	Plasticity Index (%)	
SS	5.8	1.2	7.8	57.7	43	11	8.39	2.3	60	60	SP-SM
SS+G	-	1.8	6.1	55.3	38	18	8.39	2.3	60	60	SM
JS	5.3	0.3	1.5	53.2	50	12	8.5	2.1	60	60	SP-SM
QR	1.3	0	5	50	58	15	8.37	-	60	60	SM, Well-graded sand with silt

NOTE: SS= Silty sand or sand-silt mixture, SP-SM Poorly graded sand, SP-SM Well-graded sand with silt

Table 4-4 Compression Parameters of the Soil Samples

Sample ID	Optimum Moisture Content	Maximum Dry Density
	(%)	(Mg/m ³)
South Seas (SS)	8.7	2.08
SS + 5% Bentonite	7.8	2.11
Johns	8.7	2.03
Al-Quada	8.0	2.06

data on the computer screen. During the test, constant pressure for the soil pressure (u_1) and back pressure (p_u) were supplied from two independent pressure supply sources while an overhead water tank, located at 3 m high, was used to supply the pressure, p_1 .

The water tank was also used for initial saturation of the specimen. A volume transducer was connected to the p_1 line to measure the amount of water passing through the specimen. The specimens for testing were saturated prior to testing and, e_0 was held higher than the net pressure causing a flow of the water through the specimen by at least 28 KPa during saturation as well as testing.

Preparation of test specimens. The program of permeability determination in the laboratory was carried out on specimens having a nominal diameter of 71-mm in diameter and 158-mm in height (Figure 4-6), using a backpressure, p_1 . All specimens were artificially prepared at the optimum moisture content at their corresponding maximum dry densities given in Table 4-4. The preparation method involved tamping soils in five equal layers in a standard laboratory cylindrical mold of the same nominal diameter as the specimen. The specimen preparation as well as the testing procedure followed ASTM D1584. The soil was air-dried and was then stirred as uniformly as possible. The quantity required for one layer was weighed, mixed evenly with an appropriate quantity of water, and then poured into the cylindrical mold. The soil was compressed using a metal tamper until the layer thickness, marked on the inner wall, was attained. The upper side of the soil containing the specimen was leveled off, once the whole quantity had been introduced. The specimen was then extracted from the mold and measured for true dimensions and placed on the pedestal base of the triaxial cell. The specimen ends were covered with the water-saturated filter blocks (porous stones), and a

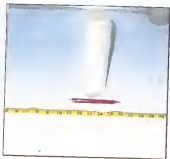


Figure 4-8 Torsional Sample of South Core (Soil)

pressure pad placed on the top of upper porous zones. Moreover, The pressure pad was connected by means of spigot tubing to the cell base as shown in Figure 4-7 to allow exit of water from the specimen. It was then enclosed in a rubber membrane and sealed tightly using an O-ring seal. The cell chamber was filled with de-aired water and the loading ram was brought just in contact to ensure that the specimen was adequately confined to rupture at the point of water exit during the time water would be flowing through it. Prior to start of the test, the specimen was saturated completely using a backpressure of 200 KPa, which was achieved in several stages to avoid possible destruction of the soil fabric by the application of a sudden high pressure. The process began with the pressure in the chamber (p_c) raised gradually to 70 KPa while the valves v_1 and v_2 both free in this stage closed. After that, port 1 was connected to the overhead water tank applying about 30 KPa. The objective of this stage was to allow water to enter the specimen from the bottom and fill the voids, expelling all of the air occupying them. When water is seen to come from the valve v_2 (this process took about 48-72 hours), the port 1 was freed from the water tank and connected to the constant pressure supply to apply a pressure of 50 KPa (p_1). After waiting for a couple of hours, both ports were closed, and the cell pressure was raised by 50 KPa, followed by a corresponding increase in p_1 by the same amount. Port p_2 was still closed and used to monitor the induced pore water pressure in the specimen due to the rise in cell pressure. The cell and backpressures were, in this manner, alternately raised, each time by 50 KPa until p_1 and p_2 reached 220 KPa and 200 KPa respectively. The reason for applying such high p_2 is to ensure full saturation of the specimen by displacing the entrapped air if any still existed. During each p_1 increment, a saturation check was made of the Skempton's pore

permeability parameter 'G' (Skempton, 1954) which was calculated as over 8.0 prior to permeability determination.

Permeability calculations. Once saturation of the specimen was ensured, port 2 was connected to the inverted tank, and valve v_1 was opened to allow the flow of water through the specimen by a net hydraulic head of $p_1 - p_2$ KPa. The elapsed time was noted using a stopwatch and the amount of water exiting from the specimen was recorded manually from the computer screen. The value of the permeability coefficient was then computed from the equation:

$$k = \frac{V_w \cdot a}{A \cdot L \cdot t + b} \quad (8)$$

where,

V_w = Volume of water flowing through the cross-section of the specimen in time (m^3)

L = Length of the specimen (m)

A = Cross-sectional area of the specimen, (m^2)

b = Hydraulic head (pressure difference) (N/m^2)

t = Elapsed time (min.)

Once the k value was determined, the specimen was put in the oven at 40-50°C to dry completely. The specimen was then wrapped with filter paper and preserved in a humidity controlled chamber to achieve saturation while a mist spray of water was applied periodically over the surface to expedite the saturation process (Figure 4-8). The alternate process of oven drying and saturation was repeated five times, and the determination of permeability was repeated following the procedure detailed above.

The volume of water flowing through the cross-section of the specimen and the elapsed time from the triaxial tests are shown in Figures 8-10 to 8-12. A period of ten



Figure 6-9 Wetting the Sample by Spraying Mist Water on the Wrapping Filter Paper

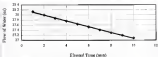


Figure 6-10. Data from Laboratory Determination of Proprietaryity on South Beach Sand

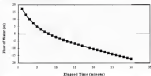


Figure 6-11. Data after Five Wetting and Drying Cycles on South Beach Sand

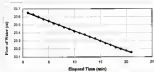


Figure 4-12. Data from Laboratory Determination of Permeability on South States Sand Mixed with 1% Bentonite

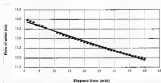


Figure 4-13. Data after Free Wetting and Drying Cycles on South States Sand Mixture with 1% Bentonite

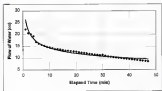


Figure 6-14 Data from Laboratory Determinations of Permeability on South River Sand

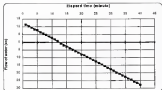


Figure 6-15 Data after Five Wetting and Drying Cycles on Idaho Sand

months was spent to finish the thermal testing program. To represent the actual field conditions in Kuwait during the different seasons, a minimum of 24-hour periods of wetting and drying cycles were completed during the tests. After the drying cycles, it was noticed that all the samples were stiffer than the prepared natural samples. The permeability results from the different tests are summarized in Table 6-3 and shown in Figure 6-16.

Falling-Head Test

Test arrangement. The laboratory determination of permeability was carried out by the falling-head permeability test using the standard composite mold permeometer (Figure 6-17). Procedures and equipment used for conducting the test followed the recommendations by Bowles (1992) and Das (1983). The area of the test mold was 61.1 cm^2 and the borehole area was 0.14 cm^2 . Furthermore, the samples were soaked in both water and chemical solutions for a minimum of 24 hours to achieve full saturation before conducting the tests.

Six samples from the Al-Qumra soil were treated by distilled water and chemical solution under a compaction to that expected in the landfill leachate in Kuwait. The samples were 165-mm in diameter and 115.5-mm in height, and they were compacted at five layers at three different water contents and at 93-95% of maximum dry density as recommended with ASTM D 1556-90. In order to investigate the soil permeability and its relation to the compaction requirement, the samples were compacted at water contents of optimum moisture content (sample A), 4% dry of optimum (sample B), and 11.6% wet of optimum (sample C). Daniel and Wu (1993) concluded after testing leachate and covers in landfill cells, that compacting the soil at a water content of $\pm 2\%$ from optimum moisture

Table 4-5 Triaxial Test Permeability Results

Name of Sample	Type of Sample	Sample Name	Permeability (cm/sec)
Scrub Slime	N	A	4.247E-07
Scrub Slime	Del N	B	2.74079E-06
SS + 2% Bentonite	N	C	9.29067E-08
SS + 2% Bentonite	Del N	D	1.433E-07
Slime	N	E	5.9034E-07
Slime	Del N	F	3.3213E-06

Legend: N = Natural, Del N = Dry and Wetting Cycles

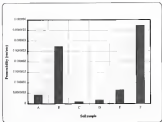


Figure 4-16. Variation of Permeability Results of Soil Tested

content and at 90 to 95 percent of maximum dry density will achieve a permeability of 1×10^{-7} cm/sec. The testing program for permeability measurements lasted for about 45 days.

Chemical analysis

The chemical solution consisted of 1,000 mg/l of each element: Phosphorus, Nitrate, and Chloride. In order to mix these elements in the solution, Potassium Phosphate Hydroxide ($K_2 HPO_4 \cdot 3H_2O$), Sodium Nitrate ($NaNO_3$), and Sodium Chloride ($NaCl$) salts were dissolved in distilled water. A minimum of 10 liters of the solution was required to perform the tests. After conducting the tests, chemical analyses were conducted on sample A that was leached by distilled water and chemical solution using Inductively Coupled Plasma Spectrometer (ICP), (Perkin Elmer Inductance Spectrometer, Plasma 400). The chemical tests were carried out to gain a clear understanding of the changes that took place, if any, in the main soil components when leached by solution other than water. The results included the percentages of Silica Oxide (SiO_2), Ferric Oxide (Fe_2O_3), Calcium Oxide (CaO), Magnesium Oxide (MgO), Cobalt Oxide (CoO), Aluminum Oxide (Al_2O_3), Sodium Oxide (Na_2O), Potassium Oxide (K_2O), Sulfate (SO_4). These elements are usually the main soil components for cemented soils in Kuwait (Al-Sabouni et al 1982, Ismail 1983, Ismail et al 1986). Chemical test results on the samples are shown in Table 4-4. The following procedure was conducted to identify soil chemical composition:

1. Place 1 gram soil sample in a glass beaker
2. Add about 20-ml of water and



Figure 6-17 Falling-Head Permeability Test Setup in Kuwait

Table 4-6 Chemical Analysis of the Soil Sample

Component	% Composition		
	Normal Test Soil	Soil Treated with Chemical Solution	Soil Treated with Distilled Water
SiO_2	45.12 \pm 1	78.88 \pm 1	78.11 \pm 1
Fe_2O_3	6.33	6.96	6.8
CaO	4.12	2.13	1.9
MgO	1.40	0.13	0.10
CuO	0.11	0.16	0.13
Al_2O_3	1.00	1.56	1.4
Na_2O	0.09	0.13	0.13
K_2O	0.13	0.19	0.13
SO_4	—	2.35	2.7

1. Add about 40-ml of Hydrochloric acid and place the beaker on a hot plate.
2. Cover with a watch glass for about one hour.
3. Remove the cover and fume the sample for 15 minutes for evaporation of the acid; the remaining solution should be about 30-ml.
4. Cool and filter (washing with de-ionized water).
5. Collect the sample into a 100-ml volumetric flask.
6. Analyze with ICP for all the elements.

In both samples leached, the concentrations of Barium (Ba), Cadmium (Cd), Chromium (Cr), Copper (Cu), Lead (Pb), Nickel (Ni), Molybdenum (Mo), Zinc (Zn) as noted were zero. In addition, the moisture content for the sample leached with chemical solution and the sample leached with distilled water were 9.53 and 8.21% respectively.

Similarly, the losses of ligand potential for the sample leached with chemical solution and the sample leached with distilled water were 4.1 and 3.7%, respectively. Results indicate hardly any change in the chemical composition.

Permeability calculation and results. In the falling-head tests, the initial head difference, h_1 at time $t = 0$ and the final head difference at time $t = t_1$ were measured on a bundle of area (a) at the same time when the water or the solution was moving through and sample of area A and length L . The permeability is determined by using the following equation.

$$k = 2.303 \frac{aL}{At} \log_{10} \frac{h_1}{h_2} \quad (9)$$

where,

a = Cross-sectional area of the standpipe (square) (cm²)

L = Length of the specimen (cm)

A = Cross-sectional area of the specimen, (cm²)

h_1 = Initial head difference at time $t = 0$ (cm)

h_2 = Final head difference at time $t = t_1$ (cm)

t = Elapsed time (sec)

The elapsed time was noted using a stopwatch, and the initial and final head differences were recorded manually. Tests were terminated when a steady state was reached. The permeability results from the different samples are summarized in Table 4-7.

Table 6-7 Filling Head Penetration Results

Leaching Fluid	Permeability (cm/yr)		
	$W_1 = 4\%$	$W_1 = 9\%$	$W_1 = 11.4\%$
Chemical Solution	7.8×10^{-6}	8.77×10^{-6}	2.3×10^{-6}
Distilled Water	4.24×10^{-7}	6.7×10^{-7}	1.8×10^{-6}

Infiltration Tests

The use of cemented sand in Korea as liners or covers has not been well investigated recently. To check the ability of these sands to prevent leachate migration with time, and to determine flow patterns, a series of infiltration tests were performed in a rigid transparent pipe. Data were collected each hour at the beginning of the test. At later stages, data were collected twice a day.

The transparent pipe was 12-in (31.5-cm) in diameter and 96-in (24.4-m) in length. The pipe was divided into three different depths: 15, 30, and 45-in, where each section represent a leachate model (Figure 6-18). Each section was filled with compacted sand from the field location. The sample soils were compacted in five layers at optimum water content and maximum dry density. Comparison tests were performed on soil samples using the ASTM D1557 procedure. A measuring pipe and a thin string were attached to record the infiltration data. Data were recorded based on average penetration at three different points on the pipe. Also, a dyed chemical solution, same as the one used in the filling-head test, was used as a leaching fluid for the tests. The quantity of the solution supplied for the 15, 30, and 45-in depth models were 400, 400, and 1000-oz

respectively. The solution quantities for the 15- and 30-cm were added in two stages, each stage at 20 liter. The second stage started after seven days of running. The pipe was sealed from bottom to prevent leakage and covered at the top to eliminate evaporation of the solution when placed on the soil.

The physical properties for the soil samples in each section are summarized in Table 4-3. Furthermore, infiltration results are shown in Figures 4-19 and 4-20.

Table 4-3 Physical Properties of Soil Samples

Pipe Depth (cm)	Sample Depth (cm)	Wet Density (Mg/m^3)	Moisture Content (%)	Dry Density (Mg/m^3)
15	12.4	2.196	4.3	2.096
30	22	2.096	4.8	1.942
45	29	2.06	4.8	1.93

Leak Testing Process

Geomembranes are not used in the construction of leach lines and sewer systems for municipal waste containment facilities in Kuwait. Nevertheless, a geomembrane was used as a leak liner in the Hazardous Waste Treatment and Reception Station (H2WTRR) for the Shuaiba Industrial Area (SIA), and in various applications such as in basement and roof linerings for expensive structures. The Kuwait Municipality future master plan for municipal waste containment facilities is proposing geomembranes as a secondary leak liner and cover. In Kuwait, the environment is an important factor when applying any material exposed to the field. The unusual high temperature in the summer and the



Figure 6-11 Sub-filtration Tests of Legionella on Datch Sample

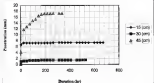


Figure 4-19 The Maximum Average Penetration Allowed in the Soil Sample

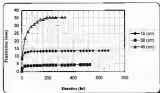


Figure 4-26 The Maximum Average Penetration Allowed in the Soil Sample

change in weather during the winter may cause critical situations in geomembrane properties. Therefore, the mechanical integrity of the geomembrane must be maintained for the design life of the landfill. Minny (1997) stated that geomembranes, unlike geotextiles, are rarely designed as structural members that carry loads, but the expected response from geomembranes is to deform due to settlement of the underlying material described as elongation, without failure.

The testing program consisted of first evaluating the static properties of the geomembrane using ASTM D631-91. Then using ASTM D3004-96, the tear resistance of the geomembrane was evaluated. Two samples were collected from two different locations. The first location, which will be described in more detail in chapter 7, was HWY285 for the Shoshone Industrial Area (Layer 1). The high-density polyethylene membrane sample was collected from the liner system of HWY285. The synthetic membrane liner system was exposed to the environment for a period of one year with no waste being disposed during this period. Although the liner was covered by a protective sand layer, five areas were not protected with no the slopes and around the leachate collection ramp. The sample was retrieved from the unprotected area. The second location was for Shoshone testing site (Layer 2) where a high-density polyethylene membrane (HDPE) was placed for a period of six months without any protective layer. Description of the Shoshone testing site will be discussed in detail in chapter 7.

All tests were conducted at a sample size of 10-80 (g/in²) and crosshead speed of 11 (mm/min). Three layer-1 specimens and four layer-2 specimens were tested following the ASTM D3004-96 test method. Furthermore, three layer-1 specimens and one layer-2 specimens were tested following the ASTM D631-91 test method. The

humidity and temperature at testing time were 30% and 23°C, respectively. For samples prepared according to ASTM D638-91, the width of layer-1 and layer-2 specimens was 6-mm (0.23-in), but the average thicknesses for layer-1 and layer-2 were 2.56-mm (0.1-in) and 1.5-mm (0.06-in) respectively. For both layer specimens the gauge length was 25.8-mm (1-in) and the grip distance was 64-mm (2.52-in). The overall length of the specimens was 115-mm (4.5-in.) For ASTM D1004-96 specimens, the width of the specimens of layer-1 and layer-2 was 12.7-mm (0.5-in) and the thickness was 2.94-mm (0.1-in) for layer-1 and 1.5-mm (0.06-in) for layer-2. Similarly, the grip distance and the specimen gauge length for both layer-1 and layer-2 were 63.8-mm (2-in) each. The overall length of the specimens was 104.8-mm (4.1-in.)

Tensile results of tensile properties (ASTM D638-91) and tear resistance (ASTM D1004-96) for layer-1 and layer-2 are shown in Table 4-9 and 4-10 respectively. The HPL testing instrument is shown in Figure 4-21 and 4-22.

Analysis and Discussion of Test Results

The purpose of the lab tests was to help in diagnosing what would be expected in field measurements of the geack permeability. Most of the tests were conducted based on field conditions and parameters. Eventually, the results will indicate the permeability characteristics of the geack samples, the effect of drying and wetting cycles on permeability, the benefits of mixing geack with bentonite, applicability of geack as a barrier, and the extent of the effect on geomembrane properties when exposed to air and climate.

Geack basic properties, shown in Table 4-1, did not achieve the minimum general requirements that were provided by Daniel and Wu (1991) for soil barrier materials.

Table 5-9 Geomembrane Tensile Properties Testing Results

Sample	Specimen Class	Peak σ_1 Area	Tensile Load Strength	Ultimate σ_1 Elongation at 0.2% yield	Stress at 0.2% yield	Young's m Modulus	
Linear 1	#	(mm ²)	(kN)	(%)	(MPa)	(MPa)	
	1	15.34	8.3172	28.43	--	19.60	181.3
	2	14.81	8.1883	27.83	--	19.45	181.3
	3	15.65	8.2897	19.88	--	18.35	144.3
	Mean	15.60	8.2189	25.73	--	19.4	180.1
	St.						
	Dev.	0.23	0.0099	7.48	--	0.68	35.8
Linear 2	1	9.80	5.2737	27.91	832.60	17.87	181.88
	2	9.80	5.2631	27.74	788.40	17.62	181.38
	3	9.80	5.2738	27.85	788.40	17.78	177.98
	4	9.80	5.2734	27.71	788.60	17.42	180.58
	Mean	9.80	5.2731	27.81	789.40	17.65	180.38
	St.						
	Dev.		0.0035	11.68	--	0.23	23.4

Legend: (Specifications)

(1) ≥ 8.324 kN for linear 1, ≥ 8.318 for linear 2(2) $\geq 600\%$ for linear 1, $\geq 800\%$ for linear 2(3) ≥ 11.4 MPa for linear 1, ≥ 18 MPa for linear 2(4) ≥ 483 MPa for linear 1, ≥ 595 MPa for linear 2

Table 6-10: Glycerol-based Tire Abrasion Testing Results

Sample	Specimen	Maximum Load	Maximum Tire Penetration	Minimum Rebound
	#	(kN)	(0.54/M)	(%)
Layer 1	1	0.2148	158.03	0.29
	2	0.2079	142.66	0.29
	3	0.2142	158.66	0.29
	Mean	0.2143	158.77	0.29
	Standard Deviation	0.0051	2.9	—
Layer 2	1	0.2126	463.26	0.2
	2	0.2145	448.46	0.2
	3	0.2168	471.46	0.2
	4	0.2146	467.26	0.2
	Mean	0.2151	464.56	0.2
	Standard Deviation	0.0053	11.00	—

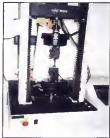


Figure 4-21 Tear Resistance Test for Leather in Kuvot



Figure 4-22 HDPE Specimen Exhibits Necking in Uniaxial Tension Test

To meet soil permeability of 1×10^{-7} cm/sec, Ghazal (1993) recommended that soil have content of a percent of fines ranges from 30 to 50, and a plasticity index between 7 to 10%. Considering gash content of clay, silt and clay percentages were between 11 and 15, all samples were non-plastic, and the silt percentages were about 80. Although this was the case, permeability results of the three natural samples compacted at optimum moisture content and maximum dry density indicated that gash can achieve low permeability levels. Jales and South Sura permeability results on natural samples were 6.9×10^{-7} and 4.23×10^{-7} cm/sec respectively.

Purchased Sodium bentonite was added to South Sura (SS) samples to improve their performance. The addition of five percent, which was the recommended practice in the U.S., was enough to lower the permeability one order of magnitude. For the bentonite and SS sand mixture, the permeability was 5.29×10^{-8} cm/sec. This previous result indicates that the addition of bentonite provided the soil with highly plastic fines, which as a result reduced the permeability of the soil. These highly plastic fines develop a thick zone of water, upon hydration, that tightly bond to the soil fines and then block the flow paths through the soil (GHA 1992).

After measuring the natural sample permeability of Jales, South Sura, and bentonite mixture with sand, the samples were then subjected to five cycles of drying and wetting. Drying and wetting of compacted gash may damage the soil by creating desiccation cracks that serve as preferential flow paths where leakage could migrate. In the field, gash tends to lose water due to the low natural soil moisture that is usually generated from high temperatures. Surface field temperatures, ranging from 40° to 50°C, were achieved in the lab for drying cycles to provide an elevated soil response. The

results, presented in Figure 4-16, point-out the effect of the cycles on the soils. Samples B, D, and F permeabilities are 2.74×10^{-8} , 1.86×10^{-8} , and 5.73×10^{-8} respectively. The results indicate that drying and wetting cycles provided a secondary flow pattern that increased the permeability by one magnitude. Based on visual inspection, no major exterior cracks were developed. Apparently, the five wetting and drying cycles were inadequate to cause damaging exterior cracks. However, interior cracks may be the major contribution for flow paths. The results may suggest that leaching of cracks after wetting was not complete. Nevertheless, after successive drying cycles, cracks, especially the interior, appear to allow flow. A recent study on compacted fractured clay showed a similar condition in which cracks did not fully heal upon hydration, but reopened during subsequent desiccation (Mollenhauer et al. 1997).

Falling head test results on the Al-Qusais soil showed that leaching a chemical solution or water through the soil can achieve the same permeability results when compacting at optimum moisture content and maximum dry density. From Table 4-7, the permeability of the soil when leached with a chemical solution was 6.73×10^{-8} cm/sec and 6.7×10^{-8} cm/sec when leached with water. Nevertheless, the results indicate that compacting goods at the wet side of optimum, and at 98% of maximum dry density, will achieve a lower permeability than compacting at dry of optimum moisture content with the same density. For a moisture content of 10.6%, the permeabilities of leached soil by chemical solution and water were 2.3×10^{-8} and 1.4×10^{-8} cm/sec respectively. For a moisture of 4%, the permeabilities of leached soil by chemical solution and water were 7.0×10^{-8} and 4.24×10^{-8} cm/sec respectively. These results suggest that compacted paths may have the same properties as compacted clay, especially those explained by the

particle interaction theory suggested by Lunin (1968). Lunin suggests that flocculated structures produced at a dry side of optimum moisture content have larger voids than those of a dispersed structure that is obtained at wet of optimum moisture content. As a result, the permeability for a flocculated structure is higher than the permeability for a dispersed structure.

The infiltration tests on compacted gash presented in Figure 6-15 and 6-20 show that penetration is initially occurring rapidly due to high swelling potential of gash and then reaching a steady state. All compacted depths of soil follow the phenomenon that was more clearly described for the 45-cm sample. The maximum penetration recorded was 70.5 mm (2.8 in) at the 45-cm sample. Most of the steady state conditions were reached after seven days of infiltration. The average maximum penetration and the average minimum penetration were 74% and 34% from the total depth of the compacted sample. These results indicate that gash resistance to leaching is reasonable compared with compacted clay. The first layer of gash, in essence, acts as a protective layer for the compacted soil.

Results from testing high-density polyethylene specimens indicate that Young's Modulus decreases considerably after long field applications. Furthermore, the strength of the specimens barely met minimum standards provided by manufacturers. The mean Young's Modulus for liner-1 was 168.1 MPa, which is 60% less than the minimum 440 MPa. Similarly, the mean Young's Modulus for liner-2 was 138.3 MPa, which is 82.4% less than the minimum 780 MPa. The average peak load for liner (1) and liner (2) was 0.1188 kN and 0.2143 kN respectively. The minimum requirements of peak load for liner-1 and liner-2 were 0.334 and 0.216 kN respectively. This supports the study

conducted by Mincy and Hsu (1992) on high-density polyethylene (HDPE). The previous study concluded that the extent of modulus and strength decreases significantly as strain rate appropriate for long-term field applications. Therefore, the ability of HDPE to elongate, without failing, is questionable in soil situations. Furthermore, the average ultimate elongation for beam-2 was 736.69%, which was below the minimum acceptable requirements. For tear resistance, the average maximum loads for beam-1 and beam-2 were 0.3342 and 0.3320 kN respectively. The maximum displacements for beam-1 and beam-2 were 0.17 and 0.2 kN respectively.

Based on these lab tests can be summarized as follows:

1. Each soil has a deficiency in a natural modulus content.
2. Each is considered a low permeability soil when compacted at optimum moisture content and maximum dry density.
3. Adding bentonite to soils decreases permeability by one magnitude compared to natural permeability.
4. Compacting at the wet side of optimum, and 95% of maximum dry density produced permeability lower than the dry side of optimum at the same density.
5. Chemical composition of the soil is nearly the same when leached by chemical solution or water.
6. Diffusion through a gash liner starts with an initial rapid increase and then turns to a steady state.
7. Young's modulus and strength of HDPE decreases after long field applications.

CHAPTER 1 FIELD TESTS

Site Description

Field tests were conducted at two sites, Sharawi Industrial Area and Sharabi Industrial Area (Figure 3-1). The Sharawi site was designated by the Kerasi Municipality (KM) to perform a Two-Stage Overhead Tests (TSO) and a long-term monitoring program on untreated garbage pads. On the other hand, the Sharabi site was investigated after receiving permission of the Sharabi Area Authority (SAA) to conduct a series of TSO tests on the latest Waste Cell located at the Solid Waste Treatment and Reception Station (SWTRS). The SWTRS is still not in operation due to management problems, although SWTRS facilities were constructed and it was ready to receive waste in 1996.

Sharawi Site

The Sharawi Industrial Area (Site 1) is located in the center of Kerasi City, within Kerasi suburbs, along the major Fourth Ring Road that extends from east to west. The area is bounded on the north by the Sharawi post, on the south by the Al-Ray area, on the east by the Al-Khalifa area, and on the west by the main area of Al-Jawra. Sharawi is famous for an extensive repair and supply industry that is associated with light construction equipment factories. Also, government ministries have many storage areas and administrative facilities that occupy parts of the Sharawi area. Moreover,



Figure 1-4 Field Notes for Louisiana

the water table in the area ranges between 4 to 5 meters below the ground surface.

The site for the testing program was located in the soccer field of the Kuwait Municipality (KM) storage complex area, which is located at the south sector of the Sharada Industrial Area. KM owns and operates three storage facilities to store its new and used equipment. The site is fenced and has one main entrance gate controlled by security guards. Furthermore, roads inside the facility are paved and in good condition.

The soccer field was about 105x60 meters and can be described as a flat area that consists of smooth mudflats close sand with sparse vegetation. KM officials designated an area of 45x40 meters for the construction of the patch pads. The selected area was located at the south part of the soccer field, and was surrounded by a paved road at one side and open areas on the other three sides.

Sharada Site

The Sharada Industrial Area (SIA), (Site 2), is located about 50 kilometers south of Kuwait City while the SWTSS is located in the northern part of the SIA, Western Sector. King Fahad Highway borders the SWTSS on the west, the National Industries Company (NIC) pipe factory on the east, Road MA-4 on the south, and the NIC sand quarry on the north. According to SIA (NIC), the water table is at depths ranging from 35 to 45 meters, with fluctuations of less than one-half meter. The water table flows to the east toward the Arabian Gulf under a gradient of approximately one-percent. Also, the surface soils are described as poorly to moderately sorted, loose to moderately compacted sand and gravelly sand. At greater depths, 35 to 40 meters, cemented soils are encountered where it overlies a limestone formation (Dammam formation). Furthermore,

The permeability of undisturbed samples extracted from the site ranged from 10^{-7} to 10^{-9} cm/sec with the permeability decreasing with increasing depth (BLA, 1992).

The testing program was conducted in the Inert Waste Cell at WWTSS. The cell is designed to receive construction debris such as damaged building material, demolition debris and inert industrial waste such as plastics, metal, wood, papers, and tires. The cell consists of a large open excavation, 1,325,000 cubic meters in volume, surrounded by a two meter wide and thirty-centimeter high berm to prevent rainwater from running into the cell. The cell is sloped at 1:3 horizontal and 1 vertical (1:3:1) and has a 100x100 meter flat bottom, graded to drain storm water to the vicinity of the base of the entrance ramp (Figure 7-2). The depth of the cell is approximately 38 meters. Moreover, the Inert Waste Cell is not lined, and no leachate collection system is installed.

Test Procedures

Site 1

Five test pads were constructed using low permeability cemented calcareous sand (Quartz) to test its applicability as a soil liner and cover. Hydraulic conductivity of each test pad was measured using a Two-Stage Borehole Permeameter (TSD). The hydraulic conductivity was assessed immediately after construction (Marsal, 1998), and at the end of the testing program (August 1998). Jarvis et al. (1987) reported the successful measurements of in situ hydraulic conductivity on compacted landfill test pads, prior to construction of the surface cap for closure, using TSD test. Previous literature indicated that pads had a high swelling potential. To overcome the swelling problem, the following solutions were recommended by Jarvis et al. (1986):

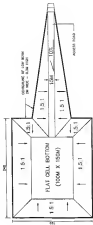


Figure 1-3 Plan of the East Nerve Cell at Solid Nerve Treatment and Reception Station

1. The Gravel should be compacted at a moisture content on the wet side of optimum
2. The fines content should be limited to 30%, and
3. Five-percent, 7%, hydrated lime should be added to the gravel layer

Drusel and Wu (1993) recommended the following design principles for liners in landfills:

1. Use rock such as sand
2. Compacted at dry or possible water content, and
3. Protect the liner with a geocomposite overlaid by top soil

Also, Drusel and Wu (1993) indicated that liners should be compacted at a water content of $\pm 2\%$ from optimum determined from modified compaction, and to a dry unit weight of at least 94-98% of the maximum value from the modified compaction. These recommendations meet three objectives:

1. Low hydraulic conductivity,
2. Low potential to shrink and crack when dried, and
3. Adequate shear strength to support structural loading.

From the cited literature, the test pads were designed and constructed to achieve low field hydraulic conductivity. Field density tests were also conducted on each pad throughout the testing program. Furthermore, temperature sensors were placed at different depths on selected pads to monitor the changes in soil temperature. The testing program for hydraulic conductivity and density tests lasted for a period of five months. On the other hand, the temperature monitoring program lasted for a period of four months. To construct the five pads, 140 m³ of clean sands, a total of eight 28 m³ trucks, and 20 m³ of fine draining sands were supplied to the site by a local company.

Test pads

Soil

Soil for the test pads was excavated and stockpiled from the Johns sand quarry two days prior to construction. Two samples were tested in the lab for soil characterization. The two samples were classified as A-2-4 by the AASHTO system, and SM in the Unified Soil Classification System (USCS). Sample 1 was composed of 6% gravel, 6% sand, and 10% silt, and clay particles, and sample 2 was composed of 6% of gravel, 80% sand, and 10% silt and clay particles (ASTM D-4002). Grain size distribution data are shown in Figure 7-3. The results of Atterberg limits tests on fractions passing the No. 40 sieve indicated that liquid limits for samples 1 and 2 were 26 and 22 respectively, and plastic limits for both samples were not determinable and therefore were recorded as non-plastic (NP) according to ASTM D4016.

Compaction tests were conducted on the soil to define the compaction curves corresponding to the modified Proctor compactive test. Results of compaction tests are shown in Figure 7-4. These compaction tests were conducted two days prior to pad construction, therefore, they were used to control construction of the test pads. The optimum moisture contents for samples 1 and 2 were 3.1 and 3.1% respectively, and the maximum dry densities were 2.8 and 2.66 g/cm³ respectively, which indicate that both samples have the same characteristics.

Specifications and construction details

All test pads were approximately 6-in long and 3-in wide. Three of the test pads were 0.61-m (2-foot) thick, and the other two pads were 0.19-m (3-foot) thick. Under each test pad, a fine-draining sand at least 0.15-m thick was placed to provide a known

hydraulic boundary condition for hydraulic conductivity computations. Plastic sheetings (25-m long, 1.25-m wide and 150- μ m thick) were placed on the pad walls to contain each pad from the surrounding environment, and to separate pads from each other.

Three test pads were constructed of three compacted 203-mm (8.0-in.) lifts to a thickness of 610-mm (24.0-in.). The other two pads had five compacted 203-mm (8.0-in.) lifts to a thickness of 1015-mm (40.0-in.). Gravel was desired to be placed in the pads at water content between optimum and $\pm 2\%$ wet of optimum, and to a dry unit weight of at least 95% of modified Proctor maximum dry unit weight (ASTM D1557). Although these were the desired requirements, it was difficult to achieve those percentages when the pads were compacted in the field. In fact, the construction contractor was directed to perform five passes on each pad, instead, at least 15 to 20 passes were performed on the pads unsaturated with water addition upon density test failure. After each five passes a density test was conducted. A field decision was made to stop compacting when reaching 95% of the maximum dry unit weight, but not at the specified water content. This was due to budget and time shortages. All the pads did fulfill the 95% of maximum dry unit weight, but at water contents ranging from 1.25% to 1.5%, percentage points dry of optimum water content.

Field measurements of water content and dry unit weight were conducted at a rate of one test per pad for the first lift of gravel, and then one test per lift for successive pads (Figure 7-5). The compaction data are summarized in Table 7-1. The first lifts for pad 1, and pad 2 were compacted by a 3-ton hand controlled, smooth-wheel vibratory roller, which was not sufficient to provide the required pressure on the soil (Figure 7-6). Consequently, all test pads were compacted using an Ingersoll-Rand SP18 smooth-wheel

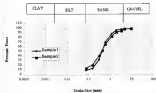


Figure 1-3 Grain-size distribution for John Samples

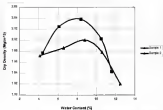


Figure 3-4 Maximum Dry Unit Weight and Optimum Water Content Measured before Construction



Figure T-5 Heavy Test Conducted on Left Tires of the Compacted Peds

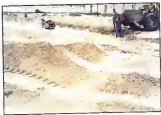


Figure 7-6 Construction Worker Operating with the 3-Tone Hand Controlled Smooth-Wheel Vibratory Roller

Table 7-1 Summary of Data Collected during Construction

Fill	Layer	Dry Density	Wet Density	Moisture Content	Degree of Compaction
#		g/cm^3	g/cm^3	%	%
1	1	1.981	2.04	7.6	82.8
2		1.934	2.06	7.8	80.7
3		1.938	2.05	6.8	82.9
4		1.911	2.04	6.8	81.5
5		1.917	2.003	4.9	82.8
1,2,3	2	1.914	2.004	4.8	82.5
4,5		1.913	2.048	7.8	82.8
1,2,3	3	1.934	2.008	7.6	89.7
4,5		1.929	2.068	6.6	86.5
4,5	4	1.934	2.048	7.8	82.8

roller (mass = 30000 kg, roller width = 4.5-m). The 30-ton roller is shown in Figure 7-7. A sheep-foot roller was not selected due to its subsequent usage in Kuwait. Six days were needed to construct all of the pads. Three construction workers participated in clearing the base and the lifts, smoothing pad edges, and mixing the soil.

Construction and Test Method

Fill 1. A rubber-tired front end loader transported sand from the stockpile to the test pad area (Figure 7-4). On the first day of placement, rain fell on the site for approximately 2 hours which added enough moisture to the sand pile. The construction workers on Fill 1 began by drying the soil, using a conventional agricultural disc. Before placing the geotextile, a fine-drumming sand underdrain 0.15-m (6-in) thick was placed without any compaction. Also, plastic sheathing was installed around the pads and was held by concrete blocks. The metal lid was placed to an approximate thickness of 8.25-m to prevent the disc and compaction equipment from mixing the underlying sand drainage layer into the geotextile.



Figure 3.7 The 18-Ton Smooth-Wheel Roller Used to Compact the Path Next to the Attached Pile of Borrowings



Figure 3-6 The Rubber-Tired Front End Loader Unit in Transport used from the Stockpile to the Test Pad Area

Compaction with the 2-ton compactor was not enough to meet specifications. Water was added to the first lift and left to equilibrate moisture conditions in the soil (Figure 7-14). After another conditioning, the first layer was compacted using the 20-ton smooth-wheel roller. Two days were needed to achieve the 95% requirement of maximum-dry unit weight.

Compaction of the second lift was completed in two days because additional water was added to the soil. The third lift was completed in one day. No additional water was added to the third lift. At the end, pad 1 had three compacted 203-mm (8.0-in.) lifts to a thickness of 610-mm (24.0-in.). The area of the test pad was 3.63 m² (39.3 ft²). Immediately after construction, the pad was covered with plastic sheeting to prevent drying or infiltration. After performing the first TCB test on the compacted soil, the plastic sheeting was removed from the pad area and the soil was exposed to the environment.

Pad 2. The first lift of pad 2 was constructed at the same time that the first lift of pad 1 was compacted. Construction workers started spreading and mixing the soil over the drainage soil layer, 615-mm thick, using a conventional agricultural disc. Because additional water was added to the lift, two days were needed to achieve the 95% requirement of maximum-dry unit weight. Compacting the second lift was completed in two days. Field moisture tests indicated that the soil was dry, therefore, the process of moisture conditioning and retransporting the soil was repeated twice to achieve the requirement for the second lift. The third lift was completed in one day. No additional water was added to the third lift.

Pad 2 had three-compacted 293-mm (8.6-in.) lifts to a thickness of 849-mm (26-ft). The area of the top pad was 5.561 meters (18.41 FT) by 5.561 meters (18.41 FT). Furthermore, the pad was covered with plastic sheeting to prevent drying or infiltration. After performing the first TSI test, the plastic sheeting was removed from the surface of the pad area. A 5.561 meters, 60-mil (1.5-mm) high-density polyethylene (HDPE) geomembrane, supplied by Al-Jalair and Rasheed Company (National Soil Company Distributor in Kuwait), was placed on the top surface of the pad for a period of four months (Figure 7-10). After this period, the condition of the pad was tested and another hydraulic conductivity test was conducted.

Pad 3. The construction workers began during the first lift using a conventional spreader/drum. After various attempts to compact the first lift, large cracks developed in the first lift. One reason to check crack developments was to examine the base of the pad and its ability to resist compressing efforts that were applied by the smooth wheel roller. The soil at the base was hydrated, possibly due to rainfall on the site since the base was exposed for two days. This pad was exposed until completing the first and second layers of pad 1 and 2. The base of pad 3 was excavated, 20-cm deep, and was replaced by the stockpiled sand at the site. The sand for another base was recommended by the site engineer and the compaction technician due to the problems in compacting the variable base. Because of the variable base, the construction of the first lift was completed in one day.

Compaction of the second lift of soil was completed in two days because additional water was added as field tests failed to achieve the specifications. The construction workers had to slow moisture conditions and recompact the second lift. One



Figure 7-9 Construction Workers Mixing Soil After Water Addition on Pad 1



Figure 7-10 High-Density Polyethylene Geomembrane Placed on Pad 2

additional day was required to complete the final lift.

Pad 3. The same dimensions and depths as Pads 1 and 2. After construction, permeability tests were conducted on the soil, and after the tests the pad was subjected to five cycles of wetting and drying. The soil was hydrated with water at low flow rates. Water was added by pouring a hose across the upper surface of the soil. The quantity of water was measured with a hand pocket, 8.1-in. in diameter and 8.45-in. in depth. Each wetting cycle consisted of approximately 750 liters of water distributed evenly over the pad. This hydration rate would correspond to an extreme rainfall event, assuming that the liner or cover would be located at the surface or would be covered by a thin protective layer. A period of at least 14-days was allowed for each drying cycle. During the drying cycles, the soil was air dried at an average temperature of 43°C (atmospheric temperature). Hydraulic conductivity was measured after the wetting and drying cycles. Also, field measurements of water content and dry unit weight (ASTM D3092 and D5937) were conducted periodically.

Pad 4. The construction for Pad 4 followed the same procedure used for Pad 1. Although no additional water was added to the lifts, the pad was completed in two days. Pad 4 consisted of four compacted 225-mm (8.9-in.) lifts to a thickness of 900-mm (3.5-ft). The area of the test was 5.641 meters (21 ft) by 7 ft. Furthermore, the pad was covered with plastic sheeting to prevent drying or leachate. After performing the first T98 test on the compacted soil, the plastic sheeting was removed from the pad area and the soil was exposed to the environment.

Pad 5. Two days were required to complete construction of Pad 5. Constructing Pad 5 followed the same procedures used in Pad 4. The pad had four compacted 225-mm

(3.5-in.) thick to a thickness of 9.0-in. (3.1-ft) and its area was 3.5-sq.-meters (116.197 ft²). After construction, a permeability test was conducted on the soil and then the pad was subjected to five cycles of wetting and drying. Wetting and drying cycles followed the same procedure that was applied for Pad 3.

Permeability measurement

Two TSB tests were conducted on each pad during the period between March 24 and August 5, 1988. Between the two tests, the compacted soil tested by TSBs was subjected to normal seasonal effects during the late winter and the complete summer season. Five TSB permeameters were installed in the pads where each pad had one TSB unit. An additional TSB permeameter sealed at the base, temperature effect gauge (TEG), was used to determine changes in the head caused by variations in temperature and barometric pressure.

The methodology for installing and conducting the TSB tests followed recommendations by Bearwell (1992) and ASTM D1586C. Equations used to reduce the data for stages 1 and 2 are described in Bearwell (1992). The first TSBs had an inside diameter of 9.1 in. (4 inch) and were set to depths between 15.4 and 27.6-cm for the 61-cm pads and to a depth of 38.3-cm for the 91-cm pads. Moreover, the extensions for stage 2 were approximately 13.3-cm long. Stage 1 and 2 sample calculation charts are presented in Table 7-2 and 7-3. The values of the vertical and horizontal hydraulic conductivity measured by TSB tests are summarized in Table 7-4.

University of Illinois at Chicago

Date (Month)	Sea Day				Tide				Wind				Remarks	
	Day	Hour	Dir.	Wind (km/h)	Dir.	Wind (km/h)	Dir.	Wind (km/h)	Dir.	Wind (km/h)	Dir.	Wind (km/h)		
June 23-28	-	41	-	143	-	-	20-3	S	-	-	-	0-10	total	
	1	21	133	133	13-181	194	203	5	133	38	0-713	0-1000	11-1	100
	2	21	133	133	13-182	194	203	5	133	38	0-713	0-1000	21-4	1800
	3	19	133	113	13-183	194	203	5	113	38	0-713	0-1000	20-10	1500
	4	17	133	124	13-184	194	203	5	124	38	0-713	0-1000	20-10	2000
	5	4	133	98	13-185	194	203	5	98	38	0-713	0-1000	20-10	total sea
	6	4	143	143	-	194	203	5	143	38	0-713	-	20-10	1800
	7	20	143	133	13-186	194	203	5	133	38	0-713	0-1000	20-10	2000
	8	20	133	114	13-187	194	203	5	114	38	0-713	0-1000	21-74	2000
	9	21	134	115	13-188	194	203	5	115	38	0-713	0-1000	20-13	2000
	10	14	133	108	13-189	194	203	5	108	38	0-713	0-1000	19-38	2000
	11	7	144	121	13-190	194	203	5	121	38	0-713	0-1000	19-38	2000
	12	0-5	161	161-5	13-191	194	203	5	161-5	38	0-713	0-1000-1	19-38	total sea
	13	21	143	143	-	194	203	5	143	38	0-713	-	19-38	2000
	14	21	143	143	13-192	194	203	5	143	38	0-713	0-1000	19-38	2000
	15	21	141	132	13-193	194	203	5	132	38	0-713	0-1000	19-13	2100
	16	21	137	118	13-194	194	203	5	118	38	0-713	0-1000	19-16	2000
	17	134	111	13-195	194	203	5	111	38	0-713	0-1000	19-16	2100	
	18	4	111	105	13-196	194	203	5	105	38	0-713	0-1000	19-16	2000
	19	4	108	148	13-197	194	203	5	108	38	0-713	0-1000	19-16	total sea
	20	20	143	143	-	194	203	5	143	38	0-713	-	19-16	2000
	21	20	143	143	-	194	203	5	143	38	0-713	-	19-16	2000

Table 7.1 Sample Chart of Stage 2 Calculations for Part 2

Item	25.1	8	111	142	327	862	207	2	327	1496	30	327	206	142	1496
	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)	1000 (mm)
100-2120	20	5	-	142	-	31	31	3	141.4	99	0.005	-	503.411	211.95	200.00
25.16 (mm)	5	20	140	121	0.0000	31	31	3	121.4	99	0.005	0.00019	111.925	240.99	
	5	20	140	121	0.0000	31	31	3	121.4	99	0.005	0.00011	106.473	241.99	
	5	12	117	146	0.0000	31	31	3	146.4	99	0.005	0.00014	109.445	220.99	
	5	5	158	167.4	0.0000	31	31	3	167.4	99	0.005	0.00019	93.146	221.99	end row
	50	50	167.4	146	-	31	31	3	147.4	99	0.005	-	155.145	210.99	End
	5	20	140	121	0.0000	31	31	3	121.4	99	0.005	0.00019	99.923	200.99	
	5	20	110	126	0.0000	31	31	3	126.4	99	0.005	0.00016	796.59	261.99	
	5	14	119	146	0.0000	31	31	3	146.4	99	0.005	0.00019	919.119	270.99	
	5	5	158	167.4	0.0000	31	31	3	167.4	99	0.005	0.00016	921.505	215.99	end row
	80	50	167.4	146	-	31	31	3	147.4	99	0.005	-	101.501	210.99	End
	5	20	140	121	0.0000	31	31	3	121.4	99	0.005	0.00019	819.929	240.99	
	5	20	120	121	0.0000	31	31	3	121.4	99	0.005	0.00019	162.945	245.99	
	5	14	120	150	0.0000	31	31	3	150.4	99	0.005	0.00019	856.931	240.99	
	5	5	116	167	0.0000	31	31	3	167.4	99	0.005	0.00014	127.445	210.99	end row
	20	20	120	146	-	31	31	3	146.4	99	0.005	-	127.445	245.99	End
	5	5	146	125	0.0000	31	31	3	125.4	99	0.005	0.00014	102.983	300.99	
	5	20	120	121	0.0000	31	31	3	121.4	99	0.005	0.00014	908.125	300.99	
	5	12	120	131	0.0000	31	31	3	131.4	99	0.005	0.00016	149.153	110.99	
	5	5	110	145	0.0000	31	31	3	145.4	99	0.005	0.00014	108.793	110.99	
	5	25	102	146	0.0000	31	31	3	146.4	99	0.005	0.00014	148.298	120.99	end row
	20	20	147	146	-	31	31	3	147.4	99	0.005	-	148.298	120.99	End

Table 7-4 Results of Vertical and Horizontal Permeabilities for the Tested Pads

Tested Area	March		August	
	K_v	K_h	K_v	K_h
	(cm/sec)	(cm/sec)	(cm/sec)	(cm/sec)
Pad 1	18.7×10^{-7}	2.47×10^{-7}	2.16×10^{-7}	2.53×10^{-7}
Pad 2	6.63×10^{-7}	7.21×10^{-7}	14.4×10^{-7}	2.96×10^{-7}
Pad 3	4.39×10^{-7}	1.33×10^{-7}	13.8×10^{-7}	2.67×10^{-7}
Pad 4	3.87×10^{-7}	2.49×10^{-7}	3.96×10^{-7}	1.91×10^{-7}
Pad 5	7.60×10^{-7}	1.86×10^{-7}	14.4×10^{-7}	1.46×10^{-7}

Soil temperatures and density monitoring

A total of nine PVC coated by thermocouple temperature sensors, 4-ft-in long with maximum temperature rating of 180°C (325°F), were installed in three connected pads at different depths. Two of the pads were exposed to the environment, Pads 1 and 4, and one pad was covered with a HDPE liner, Pad 2. Pad 4 had three sensors, S1, S2, S3 that were placed at depths of 25, 40, and 55-cm respectively. Four sensors, S5, S6, S4, and S7 were placed on Pad1 at 15, 20, 30, and 40-cm deep respectively. Pad 2 sensors, S8, and S9 were placed under the geomembrane at 20-and 40-cm depths respectively. The temperature sensor layout on each pad is shown in Figure 7-11.

The different depths and locations were selected to record the temperature variations at the three or four connected lifts. To record the soil temperatures, the sensors were connected to the digital Omega TM122B-KF hand held thermometer unit that were used in the TDS system for the permeability test. Initially, temperatures were

recorded daily for the first month and then periodically for the rest of the testing period. The soil temperature testing program was completed in four months. Tables 7-3 and 7-6 present the recorded temperatures between the period of April to August 1998 for Pads 1, 2, and 4. During the testing period, field measurements of water content and dry unit weight (ASTM D2922 and D4947) were conducted periodically. The results of both tests are reported in Table 7-3.

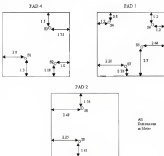


Figure 7-11 Temperature Sensors Layout on the Compacted Soil

Table 7-5 Soil Temperatures for Plot 1

Table 7-5					
Plot 1					
		Depth (cm)			
Month-Year	Date (Time)	15	30	60	90
April-98	19 (0:00 pm)	15 ^a	19	22	23
	19 (11:00 am)	25	27	28	29
	19 (2:00 pm)	30	30	30	31
	19 (3:00 pm)	30	30	31	31
	19 (3:00 pm)	31	30	30	30
	19 (11:00 am)	26	26	27	27
	20 (3:00 pm)	30	31	31	32
	20 (8:15 pm)	27	28	29	29
	20 (2:00 pm)	30	31	31	32
	21 (2:30 pm)	31	30	32	31
	21 (3:00 pm)	31	30	32	31
	26 (3:00 pm)	30	30	31	31
May-98	2 (9:15 pm)	31	31	31	31
	3 (11:30 pm)	30	31	31	32
	8 (3:45 pm)	33	33	33	34
	9 (3:30 pm)	32	30	31	31
	11 (4:15 pm)	31	30	31	31
	17 (1:30 pm)	34	33	34	34
	24 (1:00 pm)	33	30	34	33
June-98	1 (10:30 am)	19	18	17	19
	1 (3:00 pm)	25	25	26	25
	1 (7:30 pm)	34	33	30	32
	5 (1:30 pm)	34	30	17	30
	5 (6:45 pm)	37	36	32	30
	7 (2:30 pm)	39	37	34	32
	7 (3:00 pm)	36	36	36	35
	9 (2:00 pm)	37	33	30	32
	11 (6:30 pm)	35	34	34	33
	11 (6:45 pm)	30	30	26	30
	11 (8:45 pm)	30	28	28	27
	17 (6:45 pm)	39	38	36	35
	21 (8:30 am)	32	31	32	31
	21 (6:30 pm)	30	30	28	30
	24 (1:30 pm)	30	30	30	30
	27 (6:45 pm)	30	30	28	30

Table 7-5 (Continued)

PADH					
		Depth (mm)			
Month/Year	Date (Time)	15	25	50	45
June-95	25 (2:45 pm)	98	96	93	98
	26 (2:45 pm)	102	101	96	105
July-95	1 (2:57 pm)	101	103	99	104
	1 (3:58 pm)	103	101	99	103
	11 (2:46 pm)	105	101	96	100
	15 (2:28 am)	85	82	78	88
	16 (4:45 pm)	103	108	99	102
	16 (6:45 pm)	105	103	99	105
	21 (2:18 am)	90	87	86	93
	21 (2:49 pm)	100	101	99	100
	26 (2:43 pm)	102	99	97	108
	27 (1:00 pm)	106	105	101	101
	28 (2:36 am)	80	78	80	-
	28 (1:00 pm)	108	103	100	-
Aug-95	1 (2:48 pm)	106	102	101	-
	1 (1:00 pm)	106	105	104	105**
	3 (2:58 pm)	105	103	100	103
	5 (2:39 pm)	106	103	101	104
	10 (2:00 am)	79	76	81	79
	10 (2:38 pm)	96	95	100	95
	11 (2:13 am)	85	81	87	82
	11 (2:13 pm)	90	96	9	94
	11 (4:56 am)	77	76	81	88
	14 (2:30 am)	78	76	78	79
	14 (2:39 am)	76	79	83	77
	15 (1:00 am)	88	77	87	78
	16 (2:42 am)	87	82	83	89
	16 (1:00 pm)	106	99	103	106
	17 (2:00 am)	84	87	89	82
	17 (2:30 am)	84	87	89	82

* All temperatures are in Fahrenheit

** Depth was changed to 25 mm from this stage

Table 7-6 Soil Temperatures for Plot 2 and 4

		Plot 2		Plot 4		
		Depth (cm)				
		20	40	25	40	55
Month/Year	Date (Time)					
Apr-88	13-01-88 (pm)	142	84	85	87	84
	16-01-88 (am)	75	73	76	78	79
	16-07-88 (pm)	96	83	98	87	85
	17-07-88 (pm)	84	76	88	79	77
	18-07-88 (pm)	83	73	74	75	75
	19-11-88 (am)	78	74	89	88	87
	20-07-88 (pm)	80	83	83	82	86
	20-08-88 (pm)	82	89	87	87	85
	21-07-88 (pm)	96	90	90	88	85
	22-07-88 (pm)	87	88	81	78	74
	23-07-88 (pm)	93	84	83	80	77
	25-07-88 (pm)	94	85	87	84	81
May-88	3-07-88 (pm)	93	88	99	88	88
	3-07-88 (pm)	73	73	89	89	88
	8-07-88 (pm)	89	83	88	86	81
	9-08-88 (pm)	85	78	83	82	75
	11-07-88 (pm)	90	83	88	87	81
	17-07-88 (pm)	94	90	92	94	91
	24-07-88 (pm)	93	90	90	90	91
	24-07-88 (pm)	93	90	90	90	91
June-88	1-06-88 (am)	76	73	88	88	78
	1-06-88 (pm)	87	83	90	90	82
	3-07-88 (pm)	83	83	83	84	80
	5-07-88 (pm)	84	79	83	85	79
	7-06-88 (pm)	93	88	90	94	90
	7-07-88 (pm)	88	82	90	87	82
	8-08-88 (pm)	90	83	90	90	84
	9-06-88 (pm)	83	78	88	86	81
	12-07-88 (pm)	82	87	94	91	88
	13-06-88 (pm)	101	96	102	100	96
	16-06-88 (pm)	99	96	101	99	95
	17-07-88 (pm)	99	89	100	98	94
	23-07-88 (am)	87	84	84	85	85
	23-06-88 (pm)	101	95	103	101	96
	24-07-88 (pm)	102	97	101	101	95
	27-06-88 (pm)	102	96	101	100	96

Table 7-4 (Continued)

		Fm2		Fm4		
		Depth (m)				
		25	40	25	40	55
Month/Year	Date (Time)					
June-98	28 (2:45 pm)	94	81	98	97	94
	28 (3:45 pm)	100	96	100	100	96
Jul-98	5 (5:25 pm)	105	97	104	102	98
	8 (5:30 pm)	103	98	102	104	97
	18 (5:40 pm)	104	97	103	103	97
	19 (2:30 am)	87	83	85	88	86
	20 (2:45 pm)	103	93	100	98	97
	20 (3:45 pm)	103	99	103	100	99
	21 (2:30 am)	98	99	95	94	88
	22-6:45 pm	103	99	100	100	100
	26 (2:45 pm)	103	95	101	98	95
	27 (2:00 pm)	103	100	100	100	100
	28 (2:30 am)	86	83	87	92	80
	28 (2:00 pm)	104	100	103	100	99
Aug-98	2 (8:00 pm)	100	100	104	103	99
	2 (7:00 pm)	103	100	104	103	104
	5 (8:30 pm)	100	99	104	100	99
	5 (8:30 pm)	100	100	103	100	99
	10 (2:00 am)	79	78	88	81	76
	10 (2:30 pm)	98	99	104	96	99
	11 (2:15 am)	83	83	82	86	83
	11 (2:15 pm)	97	99	104	96	98
	12 (2:30 am)	79	81	85	88	87
	14 (8:00 am)	79	77	84	84	80
	14 (2:15 am)	79	80	80	80	80
	15 (2:00 am)	79	80	88	80	81
	16 (2:45 am)	86	87	90	86	87
	18 (2:00 pm)	103	100	108	98	100
	19 (2:00 am)	94	88	93	95	86
	21 (2:00 am)	98	99	100	98	99

* All temperatures are in Fahrenheit.

Table 7-1. Summary of Density and Moisture Content of Soil Peds Results

Date	Layer	Ped	Dry Density g/cm ³	Wet Density g/cm ³	Moisture Content %
April 2	3	1	1.902	2.073	9.0
April 14	3		1.876	2.068	10.0
June 23	3		1.903	2.063	9.6
July 8	3		1.893	2.088	9.9
July 28	3		1.886	2.027	7.6
July 28	2		1.923	2.093	9.3
April 2	3	2	1.947	2.116	9.0
June 23	3		1.903	2.021	6.5
July 8	3		1.921	2.087	9.0
July 28	3		1.919	2.099	9.9
July 28	2		1.902	2.093	10.2
April 2	3	3	1.909	2.027	6.4
April 14	3		1.886	2.029	6.4
June 23	3		1.917	2.076	6.2
July 8	3		1.893	1.989	5.4
July 28	3		1.904	1.976	4.1
July 28	2		1.931	2.024	7.0
April 2	4	4	1.828	1.933	6.1
April 14	4		1.872	1.983	6.2
June 23	4		1.856	1.988	6.3
July 8	4		1.901	1.993	6.0
July 28	4		1.894	1.957	4.2
July 28	3		1.938	2.046	7.0
April 2	4	5	1.842	1.956	6.2
June 23	4		1.929	2.044	6.4
July 8	4		1.909	1.986	4.2
July 28	4		1.917	1.998	4.3
July 28	3		1.929	2.063	7.0

Site 2

Four TSB tests were conducted on the Inert Waste Cell at SRTSS during the period between April 21 and May 21, 1998. The methodology for installing and conducting the TSB tests followed recommendations by Beutwell (1992) and ASTM (2002). Equipment used to enhance the data for stages 1 and 2 are described in Beutwell (1992). The four TSBs were the same parameters used to measure the permeability of the Ashland site (Figure 7-12).

The TSBs were set to a depth of 30.5-cm below the ground surface. Moreover, the extensions for stage 3 were approximately 13.24-cm long. During the testing period, field measurements of water content and dry unit weight (ASTM D6902 and D6817) were conducted at five different locations in the soil. The averages of wet and dry densities recorded in the field were 1.83 and 1.79 g/cm^3 respectively. The average moisture content recorded was 6.7%. In addition, compaction tests were conducted on the soil to define curves corresponding to the modified compactive effort (ASTM D1557). Results of compaction tests show that the optimum moisture content was 9% and the maximum dry density was 2.04 g/cm^3 . Stages 1 and 2 sample calculation sheets are provided in Tables 7-4 and 7-5. Furthermore, the values of the vertical and horizontal hydraulic conductivities measured by TSB tests are summarized in Table 7-10.

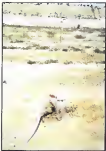


Figure 7-12 'TSH Permeameter' Installed in the Inert 'Water-Cell'

Table 7-4. *Steady Thermal Properties of Polyimides. Reprinted from Table 7-4*

Date	Time Used				Total				C	M	F	T	W	S	Total	
	0001	0002	0003	0004	0001	0002	0003	0004								
April 21, 1971 (100100)	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
April 22, 1971 (110100)	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004
	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004	0001	0002	0003	0004

Table 7-10 Results of Vertical and Horizontal Penetration for Shashu Area

TSR	K_v (cm/sec)	K_h (m/sec)
1	1.11×10^{-7}	"
2	22.9×10^{-7}	29.33×10^{-7}
3	9.9×10^{-7}	4.15×10^{-7}
4	8.21×10^{-7}	3.77×10^{-7}

Chemical Analysis

Chemical analysis were conducted on samples from area 1 and 2 using Inductively Coupled Plasma Spectrometer (ICP). (Perkin Elmer Triaxion Spectrometer, Plasma 400). The chemical tests were carried out to gain a clear understanding of the main components of the soils. The results included the percentages of Silica Oxide (SiO_2), Ferric Oxide (Fe_2O_3), Calcium Oxide (CaO), Magnesium Oxide (MgO), Aluminum Oxide (Al_2O_3), Sodium Oxide (Na_2O), and Potassium Oxide (K_2O). Furthermore, losses of ignition and moisture content percentages for all the samples are also presented in the results.

Chemical test results of samples collected from the constructed path are shown in Table 7-11. Table 7-12 presents the chemical analysis results of site 2 samples. Four samples from TSRs locations were collected at site 2. The methodology used to identify soil chemical composition followed the same procedure used in chapter 6.

In all samples, the concentration of Barium (Ba), Cadmium (Cd), Chromium (Cr), Copper (Cu), Lead (Pb), Nickel (Ni), Molybdenum (Mo), Zinc (Zn) as oxides were zero.

Table 7-11. Chemical Analysis of Peat Soil.

Component	% Composition				
	Peat 1	Peat 2	Peat 3	Peat 4	Peat 5
Si_2O_5	61.25	60.95	65.4	69.95	51.87
Fe_2O_3	0.79	0.85	0.81	0.55	0.54
Al_2O_3	1.33	1.26	1.34	1.17	1.26
Na_2O	0.28	0.83	10.052	1.43	0.83
K_2O	0.13	0.34	0.69	0.68	0.38
MgO	0.26	0.28	0.28	0.26	0.27
CaO	0.42	0.44	0.45	0.36	0.37
Moisture Content	3.86	4.39	4.9	5.14	2.65
Loss on Ignition	1.32	1.69	1.75	1.62	1.59

Table 7-12. Chemical Analysis of the Inert Waste Cell Soil.

Component	% Composition			
	TSS-1	TSS-2	TSS-3	TSS-4
Si_2O_5	67.3	66.97	69.19	66.35
Fe_2O_3	0.68	1.43	0.34	0.68
Al_2O_3	1.31	1.13	1.85	1.34
Na_2O	0.34	0.08	0.08	0.13
K_2O	0.29	0.18	0.13	0.26
MgO	1.46	1.09	0.96	1.43
CaO	3.87	1.83	1.93	3.21
Moisture Content	0.45	0.48	0.63	0.61
Loss on Ignition	4.43	3.68	4.25	5.28

Analysis and Discussion of Test Results

Results of density and moisture content conducted during test pad construction indicated that grade can be compacted at 95% to 98% of maximum dry density at moisture contents dry of optimum. Daniel and Wu (1991) recommended placing the soil at the lowest practical water content for design of liner and cover systems in and after. In the field, water addition on grade was carefully monitored due to high swelling potential, possible disturbance in the upper lifts, and possibility of a change in particle orientation. Moreover, moisture treatment of waste with flares is excess of 15 to 20 percent needs to be carefully controlled as an excess of water will result in a spongy material which is difficult to compact (MHS 1992). Following the previous recommendations and the outcome in the field, grade can be compacted relatively dry with high compressive energy, which eventually relieve residual shrinkage and desiccation cracking. Apparently, during the compaction of the lifts, it was recognized that high compressive efforts generated by the smooth roller developed longitudinal cracks that eventually sealed after further passes. These cracks were probably from breaking the orientation bonds of the grade and that consequently produce a large amount of fines capable of breaking down the bonds inside the soil.

The vertical permeability test results in the month of March showed that grade is a low permeability soil. The lowest vertical permeability was 3.87×10^{-4} , measured at Pad 4. This result suggests that additional passes applied on Pad 4, since it was the last, dense lift pad to be constructed, resulted in smaller particle sizes and more homogeneous lifts. Benson et al. (1991) reported that test pads constructed with longer hydration times and longer period of working with the soil had low permeability and fewer results. The other

pods did achieve low vertical permeability values that were 4.38×10^{-7} and 16.7×10^{-7} cm/sec for Pods 3 and 1 respectively. Pods 2 and 5 vertical permeabilities were 8.81×10^{-7} and 7.60×10^{-7} cm/sec respectively. Results of K_v did not compare well with the lab results for lab samples. The test pod permeability values differ by two magnitude higher than the lab tests. For the horizontal permeability values, results showed that K_h values were always higher than K_v values. The K_h values ranged between 1.61×10^{-5} and 2.40×10^{-5} cm/sec for Pods 3 and 4 respectively. These results indicate that gash is a natural seal since $K_h < K_v$. Also, this denotes that K_h is typically 1.5 to 4 times K_v in compacted gash soils. Daniel (1989) reported that K_h is typically 5 to 10 times K_v in compacted clay soils.

Results of vertical permeability tests in the month of August 1998 decreased for Pod 1 and increased for Pod 4. Pods 1 and 4 vertical permeabilities were 3.14×10^{-7} and 7.70×10^{-7} cm/sec. The decrease in permeability of Pod 1 implies that gash regained its cementation bonds upon desiccation. On the other hand, the increase in permeability for Pod 4 could be attributed to the depth of the pod. As the depth increases the loss of moisture from evaporation decreases, and consequently requires longer duration to escape to the surface and achieve the same moisture content levels as the 2-foot pods.

It is correct to describe the soil placed and compacted in the pods as a new soil that is different from the natural soil before excavation. The strong bonds that are generated after the long period of desiccation do breakdown after compaction and eventually cause disturbance of the soil fabric.

Results from the long-term field-monitoring program for moisture content following compaction shows that there is a moisture deficiency of the peat soil (Figure 7-10). The moisture content for Pad 1 dropped from 7% to 2.6% during the test duration. Furthermore, Pad 4 moisture contents dropped from 7% to 4.3%. On the other hand, the second and the third lifts of Pads 1 and 4 respectively preserved their moisture contents at compaction which indicates that the upper lifts are acting as a barrier (for the lower lifts) from the various environmental effects.

Several days after compacting the pads, the upper lifts were acting as hard, solid surfaces. Eventually that hardness did not affect the operation of installing TDRs due to the fact that as saturation reaches the second lift, the soil becomes wetter and easier to work. At the end of March 1994 heavy rain fell on Pads 1 and 4 for approximately ten hours. The surface of the two pads was very muddy and difficult to work. Upon repositioning the soil appeared like a dehydrated clay layer that was exposed to the sun for a long time. Later, the soil tended to create a thin crusty layer that started to shrink as more evaporation took place. It was interesting to note that the change in the soil states only affected the pad surface. When removing the crusty layer, the new surface was as solid as before. The point is that the patch surface is considered the first defense against infiltration.

For Pad 2, the vertical permeability measured in the month of August 1994 increased by one order of magnitude. K_v for Pad 2 was 1.46×10^{-6} cm/sec. Since a HDPE liner was placed on Pad 2, soil water-response rates are dissimilar to Pads 1 or 4 where the soil was exposed. The presence of the HDPE accumulation moisture at the lower surface of the HDPE and does not release it into the air. This causes the moisture to

positions again in the soil at low temperature. Results from the field moisture study illustrate the ability of the soil to preserve its moisture content or actually increase it. The moisture content of Pad 2 increased from 7% to 8% in a period of four months. The increasing permeability of soils placed under HDPE, and the statutory concerns raise some questions about using HDPE in cold regions.

The wetting and drying cycles on the selected pads, Pads 2 and 3, had soil permeabilities that compared well with lab results. Both field and lab results show that K_v increased after the wetting and drying cycles. The vertical permeability of Pad 2 increased three times more than before, and the vertical permeability of Pad 3 doubled. One reason for these different ranges of increase was the depth of Pad 3, which was larger than Pad 2 by one foot. The deeper the pad, the longer the drainage path, and the less effect were the wetting and drying cycles. K_v values for Pad 2 and Pad 3 were 12.4×10^{-7} and 44.4×10^{-7} cm/sec respectively. It was surprising to notice that there were no cracks which developed in the soil despite a series of wetting and drying cycles. Comparing these with dry of optimum in an arid region may be a key to minimizing crack development.

The horizontal permeabilities measured for all the pads in August 1998 did not change significantly from the ones measured in March 1998. K_h for all pads increased twice except for Pad 4 where K_h decreased to half. K_h values for Pads 1 and 2 were 2.33×10^{-8} and 2.86×10^{-8} cm/sec respectively. Pads 3 and 5 horizontal permeabilities were 2.47×10^{-8} and 1.44×10^{-8} cm/sec respectively. On the other hand, Pad 4 permeability was 1.48×10^{-8} . These results mean that geotext is considered a

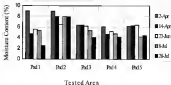


Figure T-13 Moisture Content Monitoring Data Measured from the Tested Peds

cooled soil where $K_{s1} > K_{s2}$.

Soil temperatures presented in Tables 7-3 and 7-6 point to the increased delay of the temperature wave with depth. At daily maximum surface temperatures, soil temperatures vary significantly with depth. The lowest soil temperatures were recorded at noon. During the night when minimum surface temperatures were recorded, soil temperatures reached its maximum values. Furthermore, temperature decreases as the depth below the surface increases.

Soil 3 temperatures, at 20-cm, were higher than Peds 1 and 4 in the months of April and May 1998. As the hot summer season started (June, July, and August), temperatures at 20-cm depth were slightly higher than the temperatures at the other peds. The preceding results can be explained by the fact that the HDPE geomembrane increased soil temperature by reducing evaporation and heat losses to the atmosphere, especially at night. During the summer season, the high moisture accumulation under the geomembrane that developed from the increase of outside temperature, cools down the soil to reach similar temperatures as the other peds. For the 40-cm depth temperatures recorded did not change significantly from similar depths as the other peds.

Results presented in Table 7-10 suggest that peat deposits at greater depth have low permeability values. MCS (1992) calculated field permeability rates in the order of 20% to 40% at depths 10-0 to 15.1 cm. The present results showed a low value for the coefficient of permeability which was 1.7×10^{-8} cm/sec.

The lowest permeability value was recorded at TSB 3, which was 0.2×10^{-7} cm/sec. TSBs 1, 2, and 4 vertical permeabilities were 1.11×10^{-6} , 27.9×10^{-6} and 6.21×10^{-7} cm/sec respectively. Moreover, the horizontal permeabilities for TSBs 2 and 3 were 29.28×10^{-7} and 4.13×10^{-7} cm/sec respectively. On the other hand, the horizontal permeability measured at TSB 4 was 5.77×10^{-4} cm/sec, which was slightly lower than the measured K_h . Stage 2 for TSB 1 was not completed due to the difficulties in opening the tight cap that causes the rupture of the PVC pipe and consequently affected the TSB setup.

Since the steel waste cell was not in operation, but was exposed to the environment for approximately two years, the cell experienced a series of drying and wetting cycles. The low moisture content results of the four samples point to the moisture deficiency in the soil. As a result, cracks were developed at certain areas on the surface of the cell. The depths of these cracks ranged between 3-cm (3-in) to 13-cm (5-in). Apparently the cracks were not deep enough to affect the permeability of the soil. Also, these cracks over time, mean to seal when desiccation cycles are involved. Overall, such deposits that lay at great depths may be considered as a sufficient barrier if protected from environmental deterioration by a drainage layer or similar systems.

CHAPTER 8 POLLUTE PROGRAM

Concepts and Features

Pollutev8 is a computer program that provides a solution to the one-dimensional dispersion-advection equation for a layered deposit of finite or infinite extent. The advection-dispersion equations are simplified by introducing the Laplace transform. The program falls into a well-analyzed class of models known as Finite Layer Models that are applicable to modeling hydrogeography, horizontally layered with the soil properties being the same at any horizontal location within the layer (Ravee et al. 1995). The main difference between analytical solutions and finite layer techniques is the fact that the Laplace transform in the finite layer is executed numerically rather than analytically, which allows examination of more realistic cases (Ravee 1996).

By using of finite layer solutions in Pollutev8, contaminant concentrations and the total mass that has entered can be accurately calculated at specified times and depths. Also, the accuracy of the solution does not depend on the number of layers into which the deposit is subdivided in contrast to numerical solutions, and the concentration of contaminant can be determined instantly at a specific time without prior determination of the concentration at the preceding time. The program has been used internationally for about ten years, and is currently being used in Australia, Germany, Great Britain, New Zealand, United States of America, and in Canada where the program was used in the

design of remediation landfills. For a detailed account of Polhuter's program applications and availability, refer to the Polhuter user guide by Rowe et al. (1994).

The main features of Polhuter are listed as follows:

1. Calculate the maximum concentration directly at a possible leak status or at any other depth
2. Can vary properties with time to simulate failure of an engineered system or a change in the source
3. Simulates the removal of leachate by means of a leachate collection system
4. Considers radioactive or biological decay for the source, deposit, and host
5. Can subdivide the deposit into individual layers by specifying different parameters for each layer
6. Develops a solution by assigning the bottom boundary of the deposit as impermeable, permeable, infinite, or have a constant concentration

CONTAMINANT MIGRATION

The migration of dissolved contaminants through the deposit involves different mechanisms that depend on the type of soil, degree of saturation, soil-contaminant interaction, and properties of fractures (Rowe 1994). The program calculates the migration of dissolved contaminants through the subsurface layers by transport and retention mechanisms.

Transport Mechanisms

Advection

The process by which contaminant (solute) is transported by flowing water is called advection. The mass of contaminant transported by advection per unit area per unit time, measured in a plane perpendicular to the direction of groundwater flow, is given by

$$J = u \cdot C \cdot A = q_s \cdot C \quad (7)$$

where,

α = effective porosity of the soil.

v = groundwater (average) velocity

v_0 = Darcy velocity = $\alpha \cdot v$

C_0 = concentration of the contaminant at the time of interest.

The total mass of contaminant transported from a contaminant source into a barrier at a specific time is obtained by integrating (1) with respect to time

$$m_{\text{tot}} = A \int_0^t C_0 v_0 dt \quad (2)$$

where

m_{tot} = total mass of contaminant transported

A = cross-sectional area of the landfill.

Dispersion

Dispersion is the process where the contaminant (solute) spreads out along the flow path (Srinivas and Sangreeta 1994). As a consequence, the solute will not move as a plug, like advection, but will spread and flow mix with the flow. Dispersion consists of two components, mechanical dispersion and diffusion. According to Molteni et al., (1990), mechanical mixing dominates at moderate to high groundwater velocity where diffusion dominates at low velocities.

Diffusion is the process where contaminants in the soil migrate from areas of high chemical potential to areas of low chemical potential. The mass flux transported by diffusion is proportional to the concentration gradient and can be written as

$$J = -\alpha D_v \frac{\partial C}{\partial x} \quad (3)$$

where,

n_e = effective porosity of the soil

D_e = effective-diffusion coefficient

$\frac{dc}{dx}$ = concentration gradient

The negative sign in the equation is due to the migration from high to low potential. The total mass of contaminant transported by diffusion is obtained by integrating (1) with respect to-time

$$m_d = A \int_0^t \left(-n_e D_e \frac{dc}{dx} \right) dt \quad (6)$$

where,

m_d = total mass of contaminant transported

A = cross-sectional area of the landfill.

Mechanical Dispersion is the process that involves solute mixing due to local variations in the flow velocity of the ground water. The two dispersion mechanisms can be grouped together as the Coefficient of Hydrodynamic Dispersion (2) which is given as

$$D = D_e + D_{md} \quad (7)$$

where,

D_e = effective diffusion coefficient

D_{md} = coefficient of mechanical dispersion = αv

α = dispersivity

v = groundwater (seepage) velocity

According to Gillham and Cherry (1982) and Rowe et al. (1994), the coefficient of hydrodynamic dispersion is often controlled by mechanical dispersion in sandy soils and fractured layers. Nevertheless, for velocities frequently observed in natural gradients and in well compacted liners that have permeabilities of $< 1 \times 10^{-10}$ m/s, diffusion and adsorption may both be significant where mechanical dispersion can be neglected (Rowe et al. 1995).

Retention Mechanisms

There are two types of retention mechanisms, sorption and radioactive or biological decay. Rowe et al. (1994) reported that these two mechanisms slow the migration of solute by minimizing the available mass of solute for transport.

Sorption

Rowe et al. (1994) defined sorption as the process where contaminants are removed from solution by the interaction with solid matter in the soil. This process may include cation exchange, heavy metal precipitation, and adsorption of organic solute to organic matter in the soil. Porewater can model three types of sorption: linear sorption, Freundlich non-linear sorption, and Langmuir non-linear sorption. Sorption models are described in Rowe et al. (1995, 1994).

Radioactive or Biological Decay

Radioactive and biological decays are modeled by first order decay, with the controlling parameter being the half-life of the species. The controlling equation for first order decay is written as

$$dC/dt = -\lambda C \quad e^{-\lambda t} \quad (6)$$

where,

$c(t)$ = concentration at time t ,

$c(0)$ = initial concentration,

λ = Decay constant = 0.03147/half life

Biological decay depends on the presence of suitable bacteria and substrate, and the temperature where radiotracer decay is essentially independent of the environment and is controlled by an element's atomic structure (Kjore et al. 1983)

Contaminant Migration Equations

The primary mechanisms for solute movement are advection and dispersion transport that can be written as:

$$E = uvc + uD \frac{\partial c}{\partial x} \quad (7)$$

where the parameters are those defined previously, and D is the coefficient of hydrodynamic dispersion. According to Kjore et al. (1984), the theory of contaminant migration is one-dimensional, implemented by the Fokhar's program, described by

$$n \frac{\partial c}{\partial t} + (nD \frac{\partial^2 c}{\partial x^2}) + u \tau \frac{\partial c}{\partial x} - \rho K_d \frac{\partial c}{\partial x} = uLc \quad (8)$$

where,

ρ = dry density of the soil at depth x

K_d = distribution or partitioning (sorption) coefficient at depth x

The equation simply finds the increase in contaminant concentration within a small volume which is equal to the increase in mass due to advective-dispersive transport minus the decrease in mass due to sorption and first order decay processes. The Peleusius theory is described in detail by Korte and Flecken (1945, 1957), and Korte et al. (1990).

Kuwait Environment Applications

Boundary Conditions

The soil deposit being modeled with the Peleusius program is subject to boundary conditions in the top and bottom of the deposit. The top boundary is the point of contact between the contaminant source and the soil deposit or liner system and the bottom boundary is the point of contact between the soil deposit and the less permeable strata.

The two top boundary conditions used in Kuwait case models, in the area section were constant concentration and fluxes mass. Constant concentration assumes that the top boundary maintains a constant concentration. On the other hand, the fluxes mass boundary assumes that the source concentration starts at initial value, increases linearly with time at a certain rate, and then decreases with time as the contaminant is transported to the soil and collected by the leachate collection systems if present (Korte et al. 1994). For the bottom boundary, three conditions were used in the modeled cases to represent Kuwait conditions: (a) constant concentration, (b) infinite thickness, and (c) fixed outflow velocity. Constant concentration assumes that the bottom boundary maintains a constant concentration and infinite thickness, representing that the deposit extends infinitely in depth. On the other hand, the fixed outflow velocity is applied to represent a leak aquifer

where the concentration values with time as mass is transported into the aquifer from the landfill.

Models for Landfills in Korea

To illustrate the ability of quick roads to simulate migration of contaminants from landfills, different design models were developed with Polubars using Korean environmental parameters to quickly evaluate the potential impact of these models on groundwater quality. A 300x300 meter landfill site was used as the models. The local hydrology for most models was selected to consist of a clay sand deposit overlying a gravel and sand aquifer (Figure 8-1). The clay sand deposit was assumed to have a hydraulic conductivity of 1×10^{-5} cm/sec, a porosity of 0.33, a dry density of 1.8 g/cm^3 , and a diffusion coefficient of $8.003 \text{ m}^2/\text{yr}$. The confined aquifer, consisting of gravel and sand, was 100 meters thick and had a porosity of 0.3. Although the aquifer was 100-m thick, only the upper 3 m of aquifer was modeled due to diffusion model considerations. The diffusion through the landfill cover was $1.1 \text{ m}^2/\text{yr}$.

The initial concentration after closure of the landfill was assumed to be 1,000 mg/l, and the mass of the contaminant was assumed to represent 0.2% of the waste. Furthermore, the waste was assumed to have a thickness of 7-m, and an apparent waste density of 600 kg/m^3 . For velocity calculations a 2% hydraulic gradient was used.

Reductive and biological decay rates considered for some models for a contaminant of 30 years half-life. Moreover, the contaminant was assumed to decay at

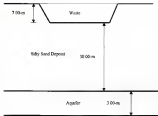


Figure 4-1. Landfill Model Dimensions for Kuwait

the base soil scores. Since all deposits consist of sand, no seepage decay was modeled. All liners consisted of 6 ft or 1-m compacted earth that had a permeability of 1×10^{-7} cm/sec. Case parameters are presented in Table 8-1.

Table 8-1 Model Case Parameters Used in Polysorb Program

Case	Top Boundary	Bottom Boundary	Liner Depth	Deposit Thickness	Aperture Thickness	Radioactive Decay
(ID)	(type)	(type)	(m)	(m)	(m)	(modeled)
1	Constant concentration	Constant concentration	0	30	100	No
2	Constant concentration	Constant concentration	0	30	100	No
3	Constant concentration	Constant concentration	0	30	100	Yes
4	Constant concentration	Fixed outflow val.	0	30	100	No
5	Finite mass source	Fixed outflow val.	0	30	100	No
6	Finite mass source	Fixed outflow val.	0	30	100	No
7	Finite mass source	Fixed outflow val.	1	31	100	No
8	Finite mass source	Fixed outflow val.	0.4	31	100	No
9	Constant concentration	Constant concentration	0.4	31	100	No
10	Constant concentration	Fixed outflow val.	1.0	31	100	No
11	Constant concentration	Fixed outflow val.	1.0	3.0	100	No
12	Constant concentration	Finite thickness	1.0	3.0	100	Yes
13 ^a	Finite mass source	Fixed outflow val.	1.0	4.0	100	Yes

Table 4-1 Continued

Case	Top Boundary	Bottom Boundary	Layer Depth	Deposit Thickness	Aquifer Thickness	Relative Disp
(ID)	Type	Type	(m)	(m)	(m)	(modeling)
(M) ¹	Finite-time event	Fixed outflow vel	0.8	40	100	Yes
(D)	Constant concentration	Fixed outflow vel	0	30	100	Yes
(T) ²²	Constant concentration	Constant concentration	1.8	-	100	No
(T) ²²	Constant concentration	Constant concentration	1.8	1.0	100	No
(T) ²²	Constant concentration	Constant concentration	1.8	-	100	No

Legend: ¹ Model with secondary landfill collection system; ²² Model 2 cases

Model 1. Basic case of pure diffusion with constant concentrations at top and bottom boundaries. The hydrology for the model was similar to the suggested model in Figure 4-1 except for the fact that the aquifer was not explicitly modeled due to the assumed zero concentration at the bottom. Contaminant migration took place through the slty sand deposit after 50 and 100 years as shown in Figures 4-2 and 4-3. The Pollinate® input data sheet is shown in Table 4-2.

Model 2. In this model, the input data file from model 1 was altered to include advective transport and a permeable base structure (aquifer). The hydrology was comprised of a 30-m thick slty sand deposit, and a 100-m thick confining, aquifer at the base. The calculated aquifer outflow velocity beneath the down gradient edge of the landfill, which corresponds to the inflow velocity at the up gradient edge plus the inflow from the landfill was 1.8 m/yr.

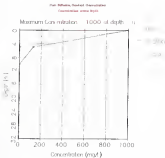


Figure 8-2 Simple Case of Pure Diffusion with Constant Concentration Boundary Condition

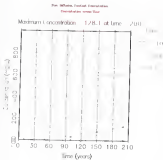


Figure 3-3 Results of Pure diffusion Case at Time 300 Years

Table 1-2 Input Data for Pure Diffusion Case with a Constant Concentration

Report Data			
Title: Pure Diffusion, with constant source and zero condensation			
No. of layers: 1			
Laplace Transform Parameters			
Time τ	0.10	Eq. 0	BCU: 2
Darcy Velocity Q	0		
Layer Data			
Layer # 1			
Ref Sub Layer	4		Fracture
Thickness	30	m	(a) None
Dry Density	1.9	g/cm^3	() 1-Dimensional
Porosity	0.32		() 2-Dimensional
Coef. of Dispersion	0.002	m^2/m	() 3-Dimensional
Distribution Coef.	0	cm^3/g	
Boundary Condition			
Top Boundary		Bottom Boundary	
() Zero Flux		() Zero Flux	
(a) Constant Conc.		(a) Constant Conc.	
() Finite Conc.		() Fixed outflow vel.	
		() Infinite Thickness	
Constant Source Conc. Boundary Condition			
Conc.		1000 mg/l	
Constant Non-Cond. Constant Boundary Condition			
Conc.		0 mg/l	

The boundary conditions and special features option for the model were divided into different cases. Case (A) had a constant concentration for top and bottom boundaries. Case (B) had the same boundary conditions as Case (A) with the addition of the special feature of radioactive and biological decay. In Case (C), the top boundary was assigned as constant concentration and the bottom boundary was selected with a fixed outflow velocity. Case (D) was similar to Case (C) as boundary conditions with the addition of the radioactive decay feature. For Case (E), which had the same input data, finite mass was the top boundary and fixed outflow velocity was the bottom boundary. The reference length of leachate that was calculated for the finite mass source boundary was 8.4-m. Results of advective-diffusive transport for the times of 20 and 200 years are shown in Figures 8-4 and 8-5.

Model 2. This model (Case E) represents the situation where the leachate mound at the base elevations was below the potentiometric surface in the aquifer. Hence, there was an upward advective flow from the aquifer to the landfill (Hydraulic Trap). For this model, the Darcy velocity into the base of the landfill was assumed to be -0.001 m/yr. The negative value for the Darcy velocity implies that the flow was upward. The base velocity used in the model was 0.4 m/d. The top boundary was a finite mass source boundary with a leachate collection system, and the bottom boundary was a fixed outflow base. Figures 8-6 and 8-7 show the results of contaminant migration at times of 20 and 200 years.

Model 3. The landfill design model consisted of a 100- or 50-cm compacted gash liner (layer 1), 3-m gash layer (layer 2), and 10-m silty sand deposit (layer 3) in most cases. The liner was assumed to have a hydraulic conductivity of 1×10^{-7} cm/sec, a

Adhesive Diffusion Transport at Time 30 Years

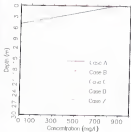


Figure B-4 Adhesive Diffusion Transport at Time 30 Years

Figure 8-5: Adhesive Diffusive Transport (Over 300 years)

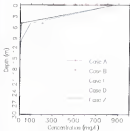


Figure 8-5: Adhesive Diffusive Transport at Time 300 Years

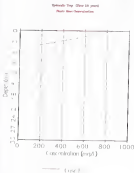


Figure 4-6: Hydraulic Trap Contaminant Migration Model at Time 50 Years

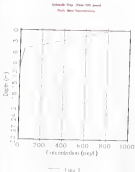


Figure 8-1. Hydraulic Trap Concentration Migration Model at Time 200 Years

porosity of 0.38, a dry density of 2.8 g/cm^3 and a diffusion coefficient of $9.63 \times 10^{-11} \text{ m}^2/\text{yr}$. The 3-m patch layer had the same characteristics as the rip-rap deposit, except it had a 0.3 porosity.

Due to the different boundary conditions, the model was divided into four cases (Case F, G, H, and I). The top and bottom boundaries for Case (F) and (G) were fixed mass sources with fixed outflow velocities. The difference between the two cases was in the layer depth where Case (F) had a 3-m primary layer and Case (G) had a 0.4-m filter. Case (H) was the same as Case (G) but its top and bottom boundaries were assigned as constant concentration. Finally, Case (I) was the same as Case (F) but its top boundary was assumed to maintain a constant concentration. Results of contaminant migration from Model 4 are presented in Figures 8-8 and 8-9.

Model 5. The model includes two cases, Case (J) and Case (K). Case (J) is similar to Case (I) with the exception of using only two layers (layers 1 and 2) instead of the three layer system. For Case (K) the lateral migration of radioactive contaminant was modeled assuming that the base of the patch layer extended to a considerable distance from the source; therefore, it was represented by an infinite thickness boundary condition. Other properties of Case (K) were the same as of Case (J). Figures 8-10 and 8-11 show the contaminant migration results at 50 and 200 years respectively. Figure 8-12 is a color concentration graph for Case (J).

Model 6. In this model a landfill with both primary and a secondary leachate collection system was modeled using Pollutant spread features. The landfill contained a fixed mass of a contaminant species and is underlain by an aquifer with fixed outflow. The liner was assumed to have a hydraulic conductivity of $1 \times 10^{-9} \text{ cm/sec}$, a porosity of

Figure 4-4: **Concentration Concentrations for Model 4 at Time 50 Years**
Initial Boundary Conditions

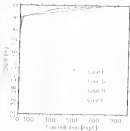


Figure 4-4: **Concentration Concentrations for Model 4 at Time 50 Years**

Continuum Model of Polymerization (Time: 100 hours)
 Different Boundary Conditions

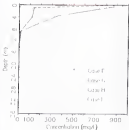


Figure 8-5 Continuum Concentrations for Model 5 at Time 100 Years

FIGURE 6-10 Model 3: For 100% Steel Bridge (Flow 100 gpm)
 Concentration in mg/L

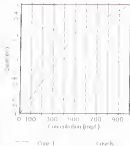


Figure 6-10 Contaminant Concentrations for Model 3 at Time 50 Years

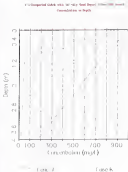


Figure 4-15: Concentration Concentration for Model 3 at Time 200 Years

Figure 9-12: Contaminant Concentration Migration for Case (B)
 Contaminant release from the well



Figure 9-12: Contaminant Concentration Migration for Case (B)

0.40, a dry density of 2.0 g/cm^3 and a diffusion coefficient of $8.02 \text{ m}^2/\text{yr}$. The secondary leachate collection system was composed of a 0.3-m thick granular layer that had a porosity of 0.40, a dry density of 1.5 g/cm^3 and a diffusion coefficient of $19 \text{ m}^2/\text{yr}$. The last layer was a 4-m, geotext layer that had the same characteristics as the silt and deposit except it had a porosity of 0.3.

The water table was assumed to be at the base of the secondary leachate collection system. Furthermore, radioactive decay was modeled for a 50 years half-life contaminant. Comparisons between contaminant concentrations for Cases L, M, and P at 50 and 100 years is shown in Figures 8-13 and 8-14 respectively.

Model 7. The model consisted of different U.S. EPA Subtitle D landfill design methods. All cases were a constant concentration source type. The first case was a Subtitle D landfill with only a primary liner and no composite liner. The primary liner consisted of a 1-m of compacted geotext. Moreover, the leachate head on the primary liner was assumed to be 0.3-m. Contaminant migration through the liner for this case is shown in Figure 8-15. In the second case, a 18-m equivalent, similar to the silt and deposit, and a leachate collector were added to case 1 (Figure 8-16).

For the third case, the landfill consisted of a composite liner and a primary leachate collection system. The composite liner was composed of a 60-mil (0.5-mm) geomembrane in good contact with a 1-m thick compacted geotext liner. Small holes with an area of 0.1 cm^2 and a frequency of 1 per acre (2.5 per hectare) were assumed for the geomembrane. The primary system was similar to the first case. Calculated concentrations for the third case are presented in Figure 8-17.

Model 55 (Primary and Secondary Treatment Reductions: Case L, 80% for all plant
 Wastewater is High)

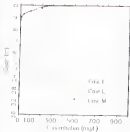


Figure B-13 Predicted Concentration-Concentrations for Model 55 at 50 years

Figure 8-14 Predicted and Boundary Condition Contour Plot, Year 100 (Base Case) (Concentration vs Depth)

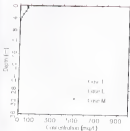


Figure 8-14 Predicted Contaminant Concentrations for Model (B) at 100 years

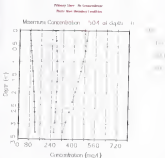


Figure 4-15 Concentrations of the Dissolved Oxygen (DO) Primary Effluent

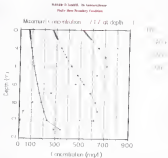


Figure 8-16 Predicted Concentrations for Subsite D Primary Layer Overlaying the Tin Boiler Aquifer.

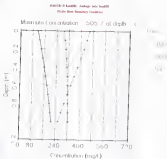


Figure 4-17 Concentrations for Subsidiary ID Composite Line and Primary Leadline Collection System.

Analysis and Discussion of Results

The Politerch computer program was an important tool in describing contaminant migration and predicting concentrations for Kurumi landfill design models. Although the predicted results may not be the exact concentrations, unless field investigations are conducted to confirm the results, the program results gave approximate concentrations that help in describing contaminant migration through the liner and then to the aquifer.

For the basic model for pure diffusion with constant source, a maximum concentration of 118.1 mg/l was predicted at a depth of 3-m after 200 years. At a depth of 10-m, the concentration was 2.43 mg/l and then declined to zero at a depth of 18.3-m.

Figures 8-4 and 8-5 discuss the advective diffusive transport where the source concentration starts at about 1,000 mg/l. In Case (IX) the concentration decreases with time as the contaminant migrates through the soil. At a 3-m depth, the maximum predicted concentration was 204.9 mg/l at 200 years for Cases (AC) and (C) where at depth of 10.5-m the contaminant concentration diminishes to almost zero. The maximum concentrations for Cases (IX) and (XI) at 3-m were 94.18 and 13.9 mg/l respectively after 200 years. At a 10-m depth, the concentrations decrease to 4.25 and 0.1 mg/l for Cases (IX) and (XI) respectively. The measured concentrations at a 3-m depth for Case (Q) were approximately zero. Considering the boundary conditions and the radioactive decay factors, Cases (C) and (Q) are suitable models for current disposal sites in Kurumi. For future landfills, Case (IX) can be considered a reasonable estimate taking into account radioactive half-life duration.

Figures 8-6 and 8-7 illustrate the concentration results at 30 and 200 years. The results indicate that concentrations migrate through the soil despite the hydraulic trap zone. At 30 years, the maximum concentration of the contaminant at 1-m was 1.54 mg/l and at 200 years it was 41.76 mg/l. Concentrations decrease to approximately zero at a 10-m depth after 200 years. Figure 8-18 shows decreasing contaminant concentration at different depths.

Model 4 results point out that boundary conditions have a major effect on the contaminant migration regardless of the layer depth. For Cases (F) and (G), since the top boundary condition was a finite mass source, the contaminant concentration at the bottom of the layer increased to a maximum value at 30 years and then decreased linearly (Figure 8-19). On the other hand, when the boundary conditions at top were constant, Cases (H) and (I), the concentration at the bottom of the layer increased gradually to a maximum point, 888 mg/l for Case (H) and 778.7 mg/l for Case (I) (Figure 8-20).

It was interesting to note that at a depth of about 10-m, concentrations had almost the same values from both boundary conditions at 200 years. These results may indicate the applicability of these systems in protecting the liner against. The average concentration from all stages in model 4 at 5-m was 13.0 mg/l.

Figures 8-10 and 8-11 show lateral migration within the deposit. Although the two layer system in Case (K) has the same top boundary conditions, constant concentration as Case (J), the selection of the bottom boundary condition did change the contaminant movement. For Case (J), concentrations at the bottom of the liner, 1-m, and the deposit, 4-m, reached a maximum value of 763.1 and 175.2 mg/l respectively at 200 years. But in the case of infinite thickness boundary condition (Case K), the

concentrations increased to a maximum value of 213.8 mg/l at the liner base after 50 years and 25.82 mg/l at the bottom of the deposit after 100 years. Later, concentrations start to decrease to a low value at 200 years. At 200 years, the 4-m deposit base concentration was as low as 18 mg/l for Case (K). The results point to the importance of a deeper cover, Case (K), in decreasing the concentration of enrichment and the aquifer role as a permeable base system in transporting the contaminants. Furthermore, although engineers do not control nature, it is important, considering the different model results, to have an adequate understanding of site geology and hydrology accompanied with contaminant transport modeling to allow confident monitoring of the site.

Figures 8-13 and 8-14 present a comparison between Case (F) that has only a primary leachate collection system, and Cases (L) and (M) that include a secondary leachate collection system. The results show that concentrations predicted for Cases (L) and (M) had lower values compared to Case (F). Two reasons are possibly responsible for lowering the concentration values for Cases (L) and (M). Apparently, the first one was due to the addition of the secondary leachate collection system, and the second was due to the radioactive decay function.

The maximum concentrations at the liner base for Cases F, L, and M were 343.5, 198.2, and 194.8 mg/l respectively after 50 years. These concentrations decrease at 200 years to 107.2, 8.10, and 7.3 mg/l respectively. Similarly, concentrations at 200 years for Case F, L, and M at the base of the deposit were 118.3, 3.8, and 2.5 mg/l respectively. Although Cases (L) and (M) have different liner designs, Figures 8-13 and 8-14 show that results for Cases (L) and (M) are almost identical.

Figures 8-13 to 8-17 illustrate Model 7 results for Polkavet's latest calculations of Subsite D Landfill. The concentrations at the base of the liner for cases 1, 2, and 3 were 204.7, 104.3, and 102.3 mg/l respectively at 100 years. At 200 years, the concentrations for cases 1, 2, and 3 were 94.3, 62.9, and 111.3 mg/l respectively at the base of the liners. It is clear from the results that leakage in composite liners overlaid by a primary liner will allow more contaminant migration to the base of the liner than a sole primary liner.

In summary, all results from the different Polkavet models indicate that patch works are adequate barriers in terms of preventing contaminant migration through the landfill and to the base aquifer, assuming that the water table is located at a sustained distance from the landfill base. At 200 years, the results indicate that the contaminant will eventually dissipate after reaching 18-in depth. Concentrations at the 18-in depth were either zero or 10% of the initial concentration as a maximum. Considering Kansas hydrology and geology, contaminants will not migrate to the base aquifer.

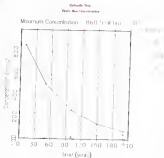


Figure 8-14 Hydraulic Trap Contaminant Concentration at Different Depths

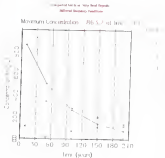


Figure 4-19 Concentration for Point Mass Source Top Boundary Condition (Case F)

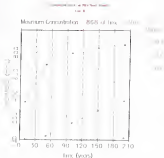


Figure 4.39. Concentration for Constant Concentration Source Top Boundary Condition (Case II)

CHAPTER 4 KUWAIT MUNICIPALITY SITE STUDY

Exposé

This Solid Waste Landfill Study is the first project to address the need for using sanitary landfill facilities for municipal solid waste (MSW) in Kuwait. Kuwait Municipality (KM), jointly with a private consulting firm and with the participation of the geotechnical group at University of Florida (GUF), conducted the project between the period of February and August 1996. The initial work for the project started in May 1997. The Department of Environmental Affairs (DEA) at KM formed a team of five members headed by the GUF member. The team made several field visits to existing landfills and conducted bench-scale tests on selected proposed landfill sites. Furthermore, the team contacted government authorities and research institutions to collect specific data such as topography, water table levels, soil types, location of water wells, and waste types and quantities etc. that were required to conduct the study.

The purpose of conducting this project was to select future solid waste landfill sites, to draft a design manual for solid waste landfill facilities, to prepare terms of reference, and then possibly tender the project. The site selection study consisted of evaluating the proposed sites in two stages. The first stage (Stage I) was a Yes or No case for each site, and the second stage (Stage II) was conducted by using the total weight selection method.

Service Areas

Initially, nine proposed landfill locations were identified in different locations of Kawerau, two in the north, three in the center, and four in the south. According to the Kawerau Municipality (1988), the project should develop a total of three landfills that would be constructed in two phases. The first phase (Phase I) would consist of two service areas with two landfills, one in central Kawerau and another one in the southern area. The second phase (Phase II) would consist of three service areas and include the addition of another landfill site located in the northern area of Kawerau. The Phase II new landfill would propose to receive waste after at least five years of establishing Phase I). Zone selections for the two phases were determined based on the Kawerau Master Plan (1997), and considering the following factors (Kawerau Municipality 1988):

1. Distribution of solid waste production based on population figures
2. Distance, time and convenience for collecting wastes
3. Locations of waste and management facilities

The northern landfill in Phase II (Zone 1) would facilitate waste collected from the north of Kawerau. The central landfill (Zone 2), which was receiving waste from the north in Phase I, would receive waste from the central areas only when the southern landfill would serve the southern areas. Figures 9-1 and 9-2 (show Phases I and II) and their service areas for Kawerau's future waste management. The Figures show that Zone 1 of Phase I would be divided to Zones 1 and 2 of Phase II and Zone 2 of Phase I will be changed to Zone 3 of Phase II. The landfills in Phase I are reported to receive equal amounts of waste, 30% of the waste generated. For Phase II, landfills located in each



Figure 6-1. Phase II of the Solid Waste I and II Study Project (A) legend from 1983



Figure 9-2. Plan II of the Bold White Landfill Study Project (Adapted from BSA 1989)

area are anticipated to receive 30% of the waste generated. Currently, the central area in Kuwait are the most populated, and consequently has a higher waste generation. The northern and southern areas in Kuwait are expected to have major developments in the future. Therefore, MSW generation in Zones 1 and 3 of Phase II would increase significantly.

Description of Proposed Landfills

Kuwait Municipality has proposed the following solid waste landfill sites:

1. Off Salwa Road
2. Off Sulaybiyah Road
3. Sakal Al-Ansari
4. Sulaybiyah
5. Al-Khradi
6. Mina Abdullah
7. Shagay
8. Uta-Qadai
9. Qadai

The first three sites, Off Salwa Road, Off Sulaybiyah Road, and Sakal Al-Ansari were included in the Kuwait Master Plan which was updated in 1997. Also, borhole tests were conducted on the first three sites in September 1997 to locate the water table and to identify soil types. Figure 5-3 shows the Ministry of Defense drilling equipment used on the borhole tests. Sites 4-6 (Sulaybiyah, Al-Khradi, and Mina Abdullah) were added later due to the fact that they are active sand quarries. As mentioned previously, Kuwait Municipality tends to consider the quarries as suitable areas for dumping wastes. The last three sites 7-9 (Shagay, Uta-Qadai, and Qadai) were selected by Kuwait

Municipality Authorities in order to consider the adjacent border area that are very far from population areas. The locations of the proposed landfill sites are shown in Figure 9-4.

4.

Al-Jubayl Road: The site is located 3 km south of Milepost 12 on Jubayl Road. A new road was constructed next to the site leading to across all fields. The site is an open area in the desert with no developments nearby (Figure 9-3). On the other side of the road, a military base and a truck field are located for sport and serve as the only substantial developments in the area. Due to its location, the site is proposed to serve the southern part of the country in Phase II.

The landslide test conducted on the site indicated that the soil is a poorly graded sand that has no plasticity, pale yellow color and a moisture content of 3.8% at one-meter depths. From 3 to 14 meter depths the soil is classified as poorly graded silty sand (SC) with moisture contents ranging from 2 to 10%. The deeper the hole the higher the moisture content values. The average bearing capacity was 340 Kg/cm^2 and the standard penetration test (SPT) values for the proposed depths ranged between 9 to 30, and generally were 20 after two meters. The maximum depth reached was ten meters with no water table encountered. At depths 5 and 9 meters, the mean diameters for soil particles were 0.5 and 0.3-mm respectively. Furthermore, according to the Unified Soil Classification, the soil at 3 meter depths had 8% gravel, 18% silt and clay, and 60% sand, and at 9 meter depths the soil had 4% gravel, 7% silt and clay, and 93% sand. Furthermore, the chloride content and total sulfates (SO_4) concentrations were 31.42 and 3,330.90-ppm respectively at 4.5-meter depths.



Figure 9-3 Ministry of Defense Drilling Equipment Used for Roadside Tests

Figure 3-4 Locations of Proposed Solid Waste Landfill Sites



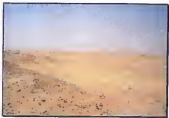


Figure 9-3 Proposed Site of Off-Salmon Road

Q12, Sakinjelele Road. The site is an open area located about 8.5 kilometers from the Seventh Ring Road. This location is in close vicinity of a semi-established residential area, agricultural area, and on the future a new urban area, according to the Master Plan of Kuwait. The current location of the site enables it to serve Zone 1 in Phase I, and Zone 2 in Phase II.

The borehole tests conducted on the site indicated that the soil is a poorly graded sand with low plasticity. Below 5 meter depth, the average moisture content was 19.70%. The average bearing capacity was 2.35 Kg/cm^2 and the standard penetration test (SPT) values ranged between 24 to 30. The maximum depth reached was 12-meters with no water table encountered. At depths of 6 and 12 meters, the mean diameter for the soil particles was 0.6-mm with a uniformity coefficient of 4.67. The soil had 0% of gravel, 0% silt and clay, and 80% sand. Moreover, the aluminum content and total sulfates (SO_4) at 6-m was 8.26 and 3,710 Tg ppm, and at 12 meter depth was 26.71 and 1053.49 ppm.

Q13, Al-Jahra Street. The site is located about 1-km from the Seventh Ring Road. This location is surrounded by construction debris from illegal dumping, which is usual practice in the Kuwait desert. Also, scattered tents and temporary structures are located next to the site, used for animal pasturing. The borehole tests conducted on the site showed that the soil is a poorly graded sand with low moisture content ranging from 7 to 9% at depth of 16 and 12-meters. The maximum depth reached was 12-meters and no water table was located. The average SPT values were 30, and the average bearing capacity was 3 Kg/cm^2 . The effective soil diameter sizes at 6 and 12-meters depth were 0.3 and 0.1-mm respectively. Similarly the mean soil diameters at 6 and 12-meters depths

were 8.7 and 8.4-mm respectively. In addition, the chloride content and total sulfates (SO_4^{2-}) were 49.8 and 2544.30 ppm respectively at 8.8-meters depth.

Al-Jubailah. The site is currently an active sand quarry that is located 8-km from the Fourth Ring Road. During quarry excavation, a small pond appeared at the bottom of the quarry, about 7-m below the ground surface (Figure 9-4). The pond was not measured and it was monitored for a period of two years. Borehole tests conducted in early July, were about 30-m from the pond, to determine the water table level. Furthermore, samples were collected and analyzed chemically from both the pond and the borehole. The tests showed that the water-table level is located about 10-m below the ground surface.

The results of water chemical analysis show a similarity in water components. Table 9-1 presents the chemical analyses conducted on water from the borehole and the pond. The Sodium content in the pond was higher than the values measured in the borehole samples. One reason may be attributed to high evaporation, which allows salts to participate and mix with water. The sulfate concentration in the borehole sample was 4,148 ppm and in the pond sample it was 7,213 ppm. Omer et al. (1981) reported that the dominant cations for the Kuwait-Group water in the southwestern area and in the middle of Kuwait, from top to bottom are: $\text{Na}^{++} > \text{Ca}^{++} > \text{Mg}^{++} > \text{K}^{+}$, and the major anion is sulfate plus chloride.

Al-Jahrahi. The site is located south of Kuwait City about 4.3-km from Travel Speed Highway (King Fahd Highway). It is currently a limestone quarry operated under the National Industries Company (NIC). The depth of the quarry ranges between 30 to 40

waters. The excavations at the Almadh quarry have exposed the Dammam Formation under five series of younger beds of the Kuwait group (Khour et al. 1994).

Ikhn, Al-Jahiliya. The site is located in an active waste disposal area that is 3 km southwest of the King Fahd Highway. This location was chosen due to former used quarry activities. No tests were conducted at this site. The site can be considered an extension of the current disposal site, and can serve Zone 1 of Phase I, and Zone 2 of Phase II.

Samraia. The site is located approximately 25 km from the Al-Wafra town, located far south of Kuwait city, in the southwest direction near the border with Saudi Arabia. It is an open area in the desert, which is very far from Kuwait City. The nearest existing road is about 16 km from the site. The site is a candidate to serve Zone 2 of Phase I, and Zone 3 of Phase II. Furthermore, a large existing water field surrounds the site from the southern direction.

Umm-Qasab. The site is located approximately 36 km in the southwest direction from Almadh City, which is located south of Kuwait City. This location is also an open desert area that is 3 km from the nearest interstate road. Moreover, the site is adjacent to a major military of Defense area. This location is so the creation of a future oil pipeline reservation that leads to a major oil field production area located south of the site.

Uthmaniyah. The site is located in the northwestern area of Kuwait, about 40 km northwest of Al-Jahiliya City next to the major power lines. The site is an open desert area with no surrounding obstructions. The nearest road to the site is about 17 km through the desert area. A physical feature map shows that large water fields are located east and west of the site.

Table 9-1 Chemical Analysis for the Wheat at Sublyngde Site

Component	Concentrations (ppm)		
	Random Sample	Field Sample	Sublyngde Soil Sample ^a
Mn	1.58	1.79	—
Pb	0.48	1.55	—
Cu	0.44 ± 0.1	1.97	768 ± 8
Mg	197.4	217.4	201 ± 8
Co	0.78	1.48	—
Al	22.8	14.5	—
Na	1921.0	4692.1	1130 ± 8
K	10.3	112.1	15.5
Ba	0.18	0.18	—
Ca	2.68	3.58	—
Ag	14.80	3.78	—
Cl	0.78	0.88	—
Cr	1.48	1.58	—
Zn	4.58	0.78	—
Fe	28.80	27.80	—
Sulfate	4594.0	7713.0	1132 ± 8

^a Adapted from Omer et al. (1987)



Figure 161. Sakaybayan Sea Pond (Adapted from RMA (1992))

Evaluation and Description of Criteria

Landfill site selection is the fundamental step in establishing an appropriate solid waste landfill. The process of selecting a site is a lengthy process that involves many considerations such as economical, environmental, socio-economical, etc.

The evaluation criteria followed for selecting landfill sites in Kuwait were divided in two stages, Stage I and II. The two stages adopted most of its items from different international regulations for solid waste management, taking into account the applicability of each item to Kuwait's environment and society.

Stage I

Stage I is the first selection stage that will reject or accept a site based on general conditions. In this stage, if the site has a 'Yes' mark then it means that the site is suitable, and the opposite is correct for a 'No' mark. However, the site is considered qualified if a disqualifying element is disregarded within a reasonable cost (Kuwait Municipality 1994). Certainly disqualifying elements vary from major to minor levels. Since this evaluation is preliminary and not final, the selected sites will be investigated fully by the contracting company for additional concerns and recommendations.

Economic, environmental and acceptability considerations were the three major criteria adopted in this stage. Economic criteria considered the cost of hauling the waste, which is directly proportional to the distance traveled. A maximum hauling distance of 30 km was considered with acceptable hauling cost. Environmental considerations include hydrological and surface water criteria. Both criteria implement certain location limitations to ensure environmental protection. The third consideration was acceptability

that includes proximity and compatibility with adjacent developments and land use as the Kuwait Master Plan. Adjacent developments include, but are not limited to oil production areas that occupy most of Kuwait's area privileged area, and military restricted areas. Stage-I evaluations for the proposed landfill sites in Kuwait are presented in Table 9-2. Although, site 4 has some major concerns about the pond location and the location of the water table level, Kuwait Municipality authority officials decided not to exclude the site due to the following reasons:

1. The site location is considered suitable to serve Zone 1 of Phase I, and Zone 2 of Phase II
2. The future landfill site can be shifted to considerable distances from the pond location

Stage II

This stage is the second site selection step in which sites that passed Stage I evaluation are evaluated extensively. The total weight method was used to determine the suitability of proposed sites. Stage II examines three important considerations: economic, environmental, and socioeconomic, and acceptability. Economic and environmental considerations are considered to represent 30% each of the total criteria. The other 40% was divided between socioeconomic and acceptability where each one had 20% of the total criteria.

Economic consideration. The first criterion considered was the hauling distance from the centroid of the solid waste generating to the landfill site, which represents 20% of the total consideration. For this criterion, the hauling distance was 50-km and the site evaluations were based on that distance. The maximum percentages were for sites that have the shortest hauling distance compared to the 50-km hauling distance. A score value

was adopted for leading distances about 50 km. Second, the need for off-site improvements, such as the cost for building or repairing an access road to the site and the cost for drainage erosion control was considered. The maximum number of points was given to sites that require minimum off-site improvement. Moreover, the distance of the proposed landfill to existing or future recycle locations was considered an important criteria. The maximum number of points was given to a site that had the shortest distance to an existing or future recycle center.

Other economic considerations were the cost of construction, operation and maintenance (O&M), and closure. The cost that was considered as evaluation basis in the previous study was the differential cost, not the base cost, which was assumed equal for all sites. For O&M costs, the weighted values for all sites were assumed the same due to insufficient site investigations. Closure cost were based on gas collection and treatment systems, visual impact reduction, and protective measures for potential wind and water erosion. Zero points were given to sites that had additional closure costs higher than other sites. Construction costs were calculated based on site conditions. Site conditions included excavation, site clearing, and soil conditions before the base system. The maximum number of points was given to a site with the maximum amount of differential cost.

Environmental consideration. This criterion includes the condition of the existing site, hydrological limitations, surface water conditions, air quality, wind erosion, and water control. The highest weight in this consideration was given for condition of existing site (20%). Regarding the previous matter, quarries have been considered outside sites to build landfill instead of regrade open areas. Consequently, sites that are

active quarries had the maximum number of points compared with sites located in open areas. Hydrological concerns such as the depth of water table below the base of a landfill, and landfill locations relative to the nearest water supply source were based on the potential impact of the landfill on groundwater. A site assigned zero points if the clearance between the proposed bottom elevation of the landfill and the seasonal high water table was less than 1.5-m. Moreover, the lowest number of points were assigned to sites that had the minimum distance to the nearest water supply.

Surface water was considered an essential aspect in the evaluation. The quality of surface water may be affected by landfills, and vice versa, landfills could be damaged by erosion. Certain key streams were considered such as one located from a tidal area, wetland -drainage courses, an active alluvial fan, and recharge zones of strategic water source aquifers. The maximum number of points was given to a site that had no potential for affecting or being affected by surface water and zero points were given to sites that do not meet the stated criteria.

Local air quality could be worse if a failure occurs in the gas collection system. Depending on the site location, points were assigned to sites based on the impact on air quality of the surrounding area. Other environmental concerns were wind erosion, which considers the direction of wind and its intensity, and vector control. Sites that are better protected from wind erosion and have limited vector impacts were given the maximum number of points.

The most important socioeconomic impact is the value of adjacent land after the landfill has been built nearby. Usually citizens reject the idea of placing landfills next to their backyard due to many social and economic problems. In the case of Karasa, the

cost may be slightly different due to the attitude of placing disposal sites far away from housing area. In the future, public awareness about landfills and their effect on the community may raise concerns about living near to the facility, which surely will affect the land value. Ideal that same land values adjacent to a landfill will remain approximately the same.

Acceptability. Acceptability has two major parts, proximity and compatibility with adjacent developments, and endorsement in the Master Plan. The items included in proximity and compatibility with adjacent developments, address how well the site integrates with present or future developments. Maximum number of points was given to sites for from key developments. Endorsement in the Master Plan considered Master Plan recommendations for all aspects concerning siting landfills. Stage II evaluation for the proposed landfill sites in Kuantan is presented in Table 6.1.

Summary

Stage II evaluation results indicate that all six sites were adequate for a landfill facility. The recommended sites for the different zones were the following:

1. Site 4 was designated for Zone 1 of Phase I and Zone 2 of Phase II. However, water table levels should be considered carefully during the design phase.
2. Site 4 was suitable to serve Zone 2 of Phase II and Zone 3 of Phase II.
3. Site 1 was approved to serve Zone 1 of Phase II. Nevertheless, the drainage course that flows near the site should be investigated during the design stage.

The study, overall, is considered the first step toward establishing the first sensory landfill in Kuwait. Table 9-4 presents the site selection results from the Stage II evaluations.

Table 9-2. Continued

Criteria	Site											
	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6	Site 7	Site 8	Site 9	Site 10	Site 11	Site 12
a. The site is within suitable areas for industrial or science corridor	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
g. The site is within areas contemplated for future oil production	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
j. Conformance to future plan	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
a. The site is within land reserved for other purposes by Kansas Municipality	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y
	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y	Y

Y/N for "No" and Y for "Yes"

Table 4-3 Design II for Landfill Best Evaluation in Kuwait

Criteria		Weight	Year 1991	Year 1992	Year 1993	Year 1994	Year 1995	Year 1996	Year 1997	Year 1998	Year 1999	Year 2000
A. Economic Consideration												
1. Distance from solid waste collection point (waste facility, dump)		30	17.4	4.6	7.8	4.4	19.4	4.3				
2. Plot for utility improvement (type and size of improvement)		10	1.6	4.4	7.8	4.4	19.4	4.3				
3. Planning to improve (sanitary and protected (sanitary services))		10	4.6	4.4	1.7	18.6	4.3	18.6				
4. Construction cost		30	1.6	18.6	17.7	19.7	19.7	19.7				
5. Operation and maintenance cost		10	1.6	1.6	1.6	1.6	1.6	1.6				
6. Closure cost		10	1.6	1.6	1.6	1.6	1.6	1.6				
B. Environment and Considerations												
7. Condition of existing site		20	6.0	1.6	6.0	1.6	19.6	19.6				
8. Anthropological												
9. Depth of natural high water table		10	16.0	18.6	16.8	1.6	18.6	18.6				
10. The horizontal distance from the edge of the landfill area to the natural water supply		10	16.0	1.7	4.3	18.6	1.7	1.7				
C. Social Issues												
a. The horizontal distance from a field or a 10-year floodplain		1	1.6	1.6	1.6	1.6	1.6	1.6				
b. The horizontal distance from a wetlands		1	1.6	1.6	1.6	1.6	1.6	1.6				
c. The horizontal distance from a drainage course		1	1.6	1.6	1.6	1.6	1.6	1.6				
d. The horizontal distance from an urban developed line		1	1.6	1.6	1.6	1.6	1.6	1.6				
e. The horizontal distance from a recharge area of drinking water source aquifers												
f. Air quality		1	6.0	1.6	1.6	1.6	1.6	1.6				
g. Noise emission		10	1.6	1.6	1.6	1.6	1.6	1.6				
h. Visual emission		10	1.6	1.6	1.6	1.6	1.6	1.6				
i. Visual emission (disturbance to birds, reptiles and mammals)		10	1.6	1.6	1.6	1.6	1.6	1.6				

Table 9-3 Continued

Criteria		Weight	Value (1991)	Score (1991)	Score (2001)	Score (2011)
		10	100	100	100	100
C. Carbon-neutral Japan		30				
1. Transportation impact			30	100	100	100
2. Adjunct land value			30	100	100	100
D. Accessibility		30				
1. Proximity and compatibility with adjacent development						
a. The horizontal distance from an all-time fault			1	1.0	1.0	1.0
b. The horizontal distance from an airport supporting jet planes			1	1.0	1.0	1.0
c. The horizontal distance from an airport supporting turbo prop planes			1	1.0	1.0	1.0
d. The horizontal distance from an adjacent property boundary			1	1.0	1.0	1.0
e. The horizontal distance from the nearest railroad, school, hospital, etc.			1	1.0	0.5	0.5
f. The horizontal distance from suitable areas due to subsidence or slides			1	1.0	1.0	1.0
g. The horizontal distance from urban congestion and/or oil production			1	1.0	0.5	0.5
h. The horizontal distance from military industrial areas			1	1.0	1.0	1.0
i. The horizontal distance from university and technologically able			1	1.0	1.0	1.0
j. The horizontal distance from university and technologically able			1	1.0	1.0	1.0
k. The horizontal distance from airport support			1	1.0	1.0	1.0
l. The horizontal distance from urban impact areas			1	1.0	1.0	1.0
2. Conformance to Master plan						
a. Suitability to Kanto Plain Plan			10	10.0	12.0	12.0

Table 9-4: Best Selection Evaluations Results for Scaled Weight Landfill Priority Project

Criteria	Weight (%)	Total Value	Site 1	Site 2	Site 3	Site 4	Site 5	Site 6
A	30	180	11,000	18.18	11,000	18,200	19,912	19,500
B	30	180	15	14.04	15,340	21.5	18,400	19,500
C	30	180	10	10	10	10	10	10
D	30	180	12,000	12.00	18,300	13,600	15,116	15,100
Total	100	-	40,705	54.22	46,710	60,180	63,184	64,600

CHAPTER 10 MINIMUM REQUIREMENTS FOR KUWAIT LANDFILLS

The purpose of this chapter is to introduce the minimum requirements for Kuwait landfill sites and to present the recommended design of liner and cover systems based on the investigations conducted in this research project. The need for these requirements is essential for several reasons listed as follows:

1. Provide an affordable environmental protection solution that depends on natural resources
2. Take immediate steps to prevent the degradation of water quality and the environment
3. Improve the standard of waste disposal in Kuwait to an environmentally acceptable level.

Since Kuwait Municipality (KMU) controls waste disposal activities and a landfill permit system for private owners is not available in Kuwait, the minimum requirements should be implemented and enforced by KMU in the design and eventual construction phases. Kuwait is a developing country and has many problems in its waste management system. Although it may have the money to construct a state-of-the-art landfill, waste disposal is not considered a high priority or the right investment of dollars by the Kuwaiti government. Very slow steps are being taken to develop immediate solutions to the waste management dilemma and its dangerous consequences on the environment and the people. Taking it to account that the Department of Environmental Affairs is eagerly trying to adopt the Solid Waste Study, the reality is that the project has a long way to

obtain government approvals. These approvals may take years to be implemented which will allow the waste problem to expand and make it even more difficult to recover.

To present a complete picture of the Kuwait waste system and its management requirements, a landfill classification system, developed by The Department of Water Affairs and Forestry (DWAF) (1994) and Elghe (1998) from South Africa, is used in the following sections. The reason for using the South Africa classification system is due to its similarity to Kuwait in climate, semi-arid area, and its waste disposal practices as a third world economy. DWAF (1994) defines the management requirements as the standards which differentiate waste disposal practices that are environmentally and publicly acceptable from those that are not. Landfills are classified according to size of operation, waste type, and potential for significant generated leachate. Where significant leachate is generated, leachate management is mandatory.

Landfill Classification

Types of Waste

Waste can be divided into two categories, General and Hazardous (DWAF 1999). Also, waste can be classified according to its biodegradable content (Elghe 1998). If the content of biodegradable material exceeds 20% by dry mass, the waste is classified as high-biodegradable waste (HB) otherwise it is low-biodegradable waste (LB).

General waste consists of domestic, commercial and dry industrial waste, and small amounts of hazardous-paste like whitebenzene and insecticides. On the other

fluid, hazardous waste that has a significant effect on public and the environment includes organic, inorganic, petroleum organic, infectious, and laboratory wastes. Moreover, hazardous waste can be further classified as extreme, high, moderate and low hazards.

Currently, Kuwait disposal sites receive many kinds of hazardous waste that are dumped with municipal solid waste (MSW). Also, the content of biodegradable content in Kuwait waste exceeds 30% by dry mass. As a result, the waste is classified as General, high-biodegradable with large amounts of hazardous quantities. This could be a conservative classification when considering future waste management. But the reality is that the new standards will take a long time to be implemented. At the same time, the improper disposal and improper waste management will continue to pose great threats to the environment and the people.

Size of Waste Stream

Size classification addresses the size of the waste stream and the management rate of the operation (ISWA² 1996). It depends on the daily rate of deposition which depends on the size of the population served. The classification is based on the maximum rate of deposition (MRD) which is the projected rate of deposition at the end of the life of the landfill. MRD is calculated from the following equation:

$$MRD = (MSW [1 + G])^t$$

where,

RD = Initial rate of deposition of waste on site at Today, assuming that the landfill is opened 3-days of the week.

D = Expected annual development rate, based on expected population growth rate in the area served by the landfill

L = Estimated life of the landfill in years

Based on the MLD calculations, the general waste landfills are divided into four size categories: Contained, Small, Medium, and Large. Table 10-1 presents the specifications for landfill size classes. Kansas landfills are classified as Large based on a 361 growth rate and 30 years expected life.

Table 10-1 Landfill Size Classification Based on MLD
(adapted from DWAAP 1994)

Landfill Size Class	Maximum Rate of Deposition (MLD) (Tons/day)
Contained (C)	< 1
Small (S)	< 25
Medium (M)	< 100
Large (L)	> 100

Landfill Generation

Landfill generation depends on the availability of water, landfill surface conditions, waste conditions, and underlying soil conditions (Sharma and Sangam 1994). The landfill generation classification system classifies landfill sites according to the extent to which they will produce leachate. The system defines sites into two types of landfill generation: sporadic, and equivalent. Sporadic leachate generation is the result of seasonal wet periods or poor site drainage (DWAAP 1994). In the case of landfills with

significant leachate generation, the landfill is controlled by other operational means rather than the installation of a costly leachate management system. On the other hand, in the case of significant leachate generation, a leachate management system would be a minimum requirement in landfill design.

To decide as to whether or not a landfill will generate significant quantities of leachate, a climatic water balance (W) is used as the classification. The climatic water balance is expressed as follows:

$$W = R - E$$

where, R is the rainfall, and E is the evaporation from the landfill cover surface (Elright 1994). For realistic and conservative assumptions run-off from the landfill surface and the ultimate storage capacity of the waste are ignored. W is calculated for the wet season of the wettest year on record. If the value of W is positive for less than one year in five for the years for which data is available, the landfill will be considered to generate sporadic leachate and the site is classified as "W". However, if the value of W is positive for more than one year in five for the years for which data is available, significant leachate will occur and the site is classified as "W".

Based on the precipitation and pan-evaporation data for the past 15 years, obtained from Meteorological Department at Kuwait International Airport, the calculated values of W were always negative even in the wettest seasons. Therefore, leachate management systems for Kuwait landfill sites are not required. Although that is true, leachate management systems are essential for current conditions due to improper

dumping and lack of regulations. It is desired to install these systems in the first sanitary landfill for conservation purposes.

Minimum Requirements for Landfills in Kuwait

Based on the previous classifications, landfills in Kuwait are classified as Q.L.W. that is, large landfill that receives general wastes and generates sporadic leachate quantities. As mentioned earlier, a Q.L.W. landfill is more representative of the current practice in Kuwait. Currently, disposed sites are badly selected, designed, and operated. Also, large quantities of high moisture wastes and liquid wastes are so disposed at the same time which results in significant leachate generation.

The minimum requirements for landfill sites in Kuwait are listed in Table 10.2. The requirements include site investigation, environmental impact assessment, and landfill design. These criteria are important steps toward establishing minimum but environmentally acceptable landfilling requirements.

Design Considerations

In developing countries, the main objective of landfill design is to provide a cost-effective and environmentally acceptable waste disposal facility. To achieve this objective, liner and cover systems must be used as barriers. Liner systems are used to minimize the infiltration of leachate into subsurface soils below the landfill thus eliminating the potential for ground water contamination, and at the same time limit the movement of landfill gases from the landfill site (Zabuhaghioun et al. 1993). Cover systems on the other hand, minimize leachate generation by minimizing water infiltration from precipitation, limit the uncontrolled release of landfill gases, and represent the

Table 19-2 Minimum Requirements for Landfills in Kuwait

Minimum Requirements	General Waste (G), Large Landfill (L)	
	Separate Leachate	Significant Leachate
A. Site Investigation	Generation (W')	Generation (W')
Define physical area of investigation	R	R
Describe topography and surface drainage	R	R
Determine surface water quality	R	R
State purpose and importance of water source	R	R
Describe man-made features	R	R
Present all relevant alternate data	R	R
Describe vegetation existing on site	R	R
A.1 Sub-Surface Features		
Identify first or most boundaries	R	R
Soil data and characteristics	R	R
Soils permeability tests	R	R
A.2 Geology		
Describe stratigraphy and lithology	R	R
Identify seismic hazards	R	R
A.3 Geohydrology		
Determine ground water flow and depth	R	R
Determine ground water quality	R	R
Ground water usage	R	R
Investigation of aquifers	R	R
Appropriate pump testing	R	R
Soil leachate and surface subsidence	R	R
A.4 Investigate Potential Gas Hazards	R	R
B. Environmental Impact Assessment	R	R
Environmental consequences of failure	R	R
Prepare action plan	R	R
Environmental impact control report	R	R
C. Landfill Design		
C.1 Conceptual Design		
Confirm site characteristics	R	R
Assess water volume	R	R
Indicate unsaturated zone water level		
excavation	R	R
Determine available surface	R	R
Estimate storage utilization	R	R

Table 10-2 Continued

Minimum Requirements	Critical Waste (C), Large Landfill (L)	
	Spillable Leachate	Significant Leachate
C. LA. Site layout design		
Surface hydrology and drainage design	R	R
Development and layout plan	R	R
Closure/Rehabilitation plan	R	R
Design of leachate management system	R	R
Monitoring system design	R	R
C.1B. Preliminary Closure Plan		
Final site plan	R	R
Landscaping plan	R	R
C.2. Testing of Soils and Materials	R	R
D. Technical Design		
Surface hydrology and drainage design	R	R
Water quality monitoring system	R	R
Leachate management and monitoring system	R	R
Gas management and monitoring system	R	R
Slope stability	R	R
Excavation control design	R	R
D.1. Lining system		
Compacted base layer	R	R
Compacted GCLs base	R	R
Leachate collection layer	NR	R
Protecting geotextile	NR	R
Leachate distribution layer and under layer	NR	R
Second GCL & filter layer	NR	NR
Final isolation layer	NR	NR
Flexible membrane liner (FML)	NR	NR
D.2. Final cover design		
Layer of gravel	R	R
Compacted GCLs layer	R	R
Geotextile layer	NR	R
Drainage layer	NR	R

R: Required, NR: Not required

granulation of various (Sharma and Sanghvi 1974, Techtongyorn et al. 1981). Layer and cover systems consist of different components that serve the purposes of the landfill design method.

Layer System

A suitable liner system for sanitary landfills consists of two main element layers, that is a leachate collection layer and a compacted geotext layer. The recommended liner elements are depicted in Figure 10-1, and the details and functions associated with each layer are described below as follows:

- | | |
|---------|---|
| Layer A | A 400-mm thick pervious sand layer that serves to filter the leachate before it is collected for treatment, and to protect the leachate collection pipes. |
| Layer B | A geotextile filter layer that is used to maintain the interlocking of Layer A and Layer C. |
| Layer C | A leachate collection layer comprising of a 100-mm thick layer of gravel or sand. The layer serves as a collection and drainage layer for any leachate that may be generated within the landfill. |
| Layer D | Four layers each of a 25-mm thick geotext liner layer that must be compacted at a water content of 2% from the optimum value and at a dry unit weight of at least 93-95% of the maximum value determined from the modified compression test (ASTM D1557). The permeability of the liner system must not exceed 1×10^{-7} cm/sec. Also, interfaces between D layers must be tightly sealed to assist in handling the layers together (LWACF 1996). Moreover the surface of every geotext liner layer must be graded at a minimum gradient of 2% toward the leachate collection ramps. |
| Layer E | An ultimate thickness layer of an-ole red that has a minimum permeability of 1×10^{-7} cm/sec. |

Soils that need extra limitations on leachate movement due to high potential for groundwater contamination have to add three optional layers (F, G, and H) below the barrier layer (Layer D). Layer F consists of geotextile that protects Layer G from contamination by fine materials from Layer D. Layer G is a leakage detection and collection layer that has a minimum thickness of 150-mm. Moreover, Layer H is a 300-mm base preparation layer of compacted virgin soil that is constructed to the same compacted standards as Layer D. The previous layers, i.e., Layers F, G, and H, can be eliminated with an adequate leachate monitoring system. It is suggested that downstream monitoring wells could be used, provided they are observed periodically.

Cover System

In a double barrier to the liner system, the cover system is made up of a series of layers that each has special functions. Figure 10-2 shows the recommended components of the cover system. The details and variations associated with each layer is described below as follows:

- | | |
|---------|--|
| Layer J | A 300-mm thick layer of silt and natural gravel mixture. The slope of the layer must be graded initially at a minimum of 2%, after accounting for settlement. |
| Layer K | A 300-mm thick layer of sand or gravel that has a maximum slope of 2% at the bottom of the layer. The hydraulic conductivity of the drainage layer should not be less than 1×10^{-1} cm/sec. The layer should be constructed to ensure that allow any uncollected seepage water to be transferred to a leakage collection sump (DWAJ 1994). |
| Layer L | Three layers each of a 300 mm thick geotextile barrier layer that must be compacted at a water content $\pm 2\%$ from the optimum value and to a dry unit weight of at least 95-98% of the maximum value determined from the modified compaction test (ASTM D1557). The permeability of the barrier system must not exceed 2×10^{-11} cm/sec. |

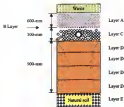


Figure 10-1. Liner System Design for Konrad Municipal Solid Waste Landfill

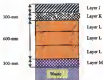


Figure 16-2 Cover Design for Sanitary Municipal Solid Waste Landfill

Moreover the surface of every patch cover layer must be graded at a minimum gradient of 2% to drain off precipitation.

Layer M A. 100-mm thick natural soil gas collection and ventilation layer

CHAPTER 11 CONCLUSIONS

Current municipal solid waste (MSW) disposal management practices in the State of Kuwait create a great threat to the country's environment and human health. Current disposal sites do not meet the minimum safe criteria for MSW that will ensure the protection of human health and the environment. Poor liner and cover systems are not included in the current practice of landfill dumps. Moreover, lack of minimum requirements to build sanitary landfills is the current dilemma for Kuwait Municipality.

Evaluation of current disposal sites indicated that all of the sites were not designed for dumping waste, and were operated poorly with inefficient equipment. Also, most of the sites receive all kinds of waste in addition to unacceptable quantities of hazardous wastes. Also, there are no monitoring systems. A review of two case studies from old sites showed high concentrations of leachate components, and HIGH explosive levels for methane gas in selected locations.

The principal objective of this research was to help KMD to build safe MSW sites that are environmentally acceptable yet cost-effective. To achieve this objective, lab and field tests along with the Fathareh computer program, were used to study the applicability of using the natural clayey layer of sand (Qatifa) as liner and cover systems. Also, High-Density Polyethylene (HDPE) liners were tested in the lab and in the field to check their suitability for the Kuwait environment.

Three samples from different parts of the country were tested in the lab for permeability and other properties. The results indicated that gash is a low permeability soil when compared at optimum moisture content and maximum dry density. Furthermore, gash compacted on the wet side of optimum, and 80% of maximum dry density, produces permeability lower than the dry side of optimum at the same density. Nevertheless, results from the five test pads constructed show that gash can be compacted at 55% to 80% of maximum dry density and at moisture contents above dry of optimum. At that specific range, vertical permeabilities were as low as 2.8×10^{-7} cm/sec. Drying and wetting cycles on the gash samples and pads increased the permeability by one magnitude, which may be attributed to a secondary flow pattern. Cracks were not developed in the field, which suggest that compacting dry of optimum in an arid region may minimize crack development.

Permeability in the field was evaluated by a Two-Stage Barbiere test (TSB). The permeability values for the gash deposit, located 15cm below the surface, ranged from 8.3×10^{-7} to 22.9×10^{-7} cm/sec. In the constructed test pads the permeability values ranged from 1.0×10^{-7} to 14.7×10^{-7} cm/sec immediately after construction, and after the wet, monthly period permeabilities were between 1.9×10^{-7} and 14.4×10^{-7} cm/sec. It was concluded that a gash deposit at great depths is considered a secondary barrier system in the liner design. At the test pads, gash upon desiccation required no construction bonds and decreased its permeability as much as one magnitude. As a result, it is recommended that there and cover should be constructed between spring and summer seasons to allow sufficient time for desiccation.

Similarly, results from the different Polutant models describing contaminant migration and predicting its concentration, indicated that gash seals are adequate barriers. Contaminant results show that concentrations at a 10-m depth were either zero or at maximum, 10% of initial concentration after 200 years.

Samples of HDPE liners were tested for their tensile properties and low resistance. Results from HDPE liners exposed for a period of one year and six months, showed that Young's Modulus and strength of the liner decreases after long field applications. In the field, HDPE tends to accumulate moisture due to high temperatures, which eventually increased the permeability of the gash pad. This was particularly true for those systems which had been in place for a minimum of six months after construction. Based on permeability results and slurriness concerns, HDPE should not be used in Kuwait for MSW landfills.

In an effort to study the variation of temperature at the seal pad layers, temperature sensors were installed at different depths in the pads. It was clear that soil temperatures vary with depth except for the first 40-cm in depth where temperatures recorded did not change significantly.

Overall, compacted gash can be used as liner and cover systems in Kuwait. The gash layer must be compacted at a water content $\pm 2.5\%$ from the optimum value and to a dry unit weight of at least 95-98% of the maximum value determined from the modified compaction test (ASTM D1557) which will achieve low permeability. A desiccation period of 3-6 months is recommended for the systems during the dry season to decrease permeability further. No HDPE liner is recommended. Nevertheless a leachate collection

system (LCS) is required for the sites even though the Karwai site classification system does not require the installation of LCSs. Also, numerous requirements for site investigation, environmental impact assessment, and landfill design that include monitoring systems should be implemented to establish environmentally acceptable landfill.

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BIOGRAPHICAL SKETCH

Amrute Al-Taqout was born in Kuwait on September 17, 1948. He graduated from Youssef Ibn Ezzah High School, Kuwait, in 1967.

He was granted a scholarship from the Ministry of Education of the State of Kuwait to earn a bachelor of science degree in architectural engineering at the United States of America, which he received from the University of Miami, Coral Gables, Florida, in 1969. He was awarded a Bachelor of Science in Architectural and Civil Engineering in 1993.

He worked in the Ministry of Public Works in Kuwait for eight months as a site engineer. He was granted a scholarship from the University of Kuwait to complete his master's and Ph.D. degrees in civil engineering at the United States of America. In the spring of 1993, he enrolled in the University of Florida graduate school in the Department of Civil Engineering. In Summer 1996, he was awarded Master in Civil Engineering. He started his Ph.D. program in Fall 1996.

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.


Frank C. Townsend, Chairman
Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.


David G. Thompson
Associate Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.


Paul Kraft
Associate Professor of Civil Engineering

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.


Timothy W. Townsend
Associate Professor of Environmental Engineering Science

I certify that I have read this study and that in my opinion it conforms to acceptable standards of scholarly presentation and is fully adequate, in scope and quality, as a dissertation for the degree of Doctor of Philosophy.


Paul Thompson
Professor of Civil Engineering

This dissertation was submitted to the Graduate Faculty of the College of Engineering and the Graduate School and was accepted as partial fulfillment of the requirements for the degree of Doctor of Philosophy

August 1999


Richard M. Phillips
Dean, College of Engineering

M. J. Oleson
Dean, Graduate School